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**GEOTECHNICAL ENGINEERING REPORT**
**RENO RAILROAD CORRIDOR DRAFT EIS**
**RENO, NEVADA**

Kleinfelder Project Number:  30-1307-10.001

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# EXECUTIVE SUMMARY

## I. Introduction and Background

This report contains the geotechnical data and a discussion of geotechnical design issues and construction methods for the Reno Railroad Corridor. This report serves as a source document for subsurface information regarding soils conditions, groundwater conditions and environmental impairment to both the soil and groundwater system. Kleinfelder performed this study as part of the Preliminary Engineering and Environmental Impact Statement (PE/EIS) scope of services currently underway by the Nolte team. This document provides the baseline information from which further engineering assessments will be developed as the preliminary engineering effort continues.

The Reno Railroad Corridor Project is conceived to improve traffic circulation, air quality and public safety by reducing or eliminating existing conflicts between the Union Pacific Railroad and the street system in downtown Reno. The EIS scoping process has considered numerous alternatives within the existing railroad right-of-way, along parallel alignments within a few blocks of the existing alignment, and routes which bypass the entire Reno downtown area. This geotechnical report focuses on conditions within the downtown area, since geotechnical conditions will have greater influence on alternatives involving deep cuts (trench or tunnel alternatives). More detailed and specific studies will likely be needed to provide geotechnical information suitable for engineering design. Therefore, greater emphasis will be placed on these alternatives in this report.

## II. Summary of Subsurface Conditions

Our exploration revealed that the soils along the alignment can be broken down into two basic groups. The upper soils consist of undifferentiated finer-grained flood deposits and man made fills. These upper soils vary from 2 feet to 18 feet thick. The deeper soils consist of very coarse-grained glacial outwash materials. These deeper soils consist of heterogeneous mixtures of sands, gravels, cobbles and boulders. Fines (percent passing No. 200 Sieve) contents vary greatly and fines consist generally of low plasticity silts and clays. In general, the deeper soils exhibit high shear strength, moderate to very high permeability and low compressibility characteristics.

When drilling was completed in late Summer, 1999, groundwater was encountered at 14 to 31 feet below the existing ground surface, corresponding to a groundwater elevation of approximately 4500 (feet above Mean Sea Level) at the west end of the study area, to approximately 4455 at the east end. Subsequent measurements through April, 2000 show highest water levels in March, which were generally on the order of 2 feet to 4 feet higher than the late summer months. This will place the central approximately 3,500 feet of the downtown core area below the seasonal high water table, for the trench and tunnel alternatives. The ends of the study area, west of Ralston Street and east of the Sierra Pacific Power Company substation between Evans and Wells Avenues, will likely not encounter groundwater.

Most soils above the groundwater have been contaminated by petroleum hydrocarbons. Nineteen of thirty samples analyzed for this study contained fuel or oil derivatives. Although five samples contained concentrations above 100 mg/kg which would require treatment and disposal offsite, the remaining samples tested lower levels and can likely be reused as fill soils. Screening and segregation of contaminated soils will be necessary during construction activities.

Much of the groundwater below the site contains dissolved petroleum hydrocarbons and volatile organic compounds (VOCs). Five of nineteen groundwater samples analyzed for total petroleum hydrocarbons contained dissolved petroleum, all in concentrations less than 10 mg/l. Eighteen of the nineteen groundwater samples tested contained chlorinated VOCs, although no sample concentrations exceeded 20 µg/l. These compounds appear to be widely dispersed throughout downtown Reno. No free floating petroleum product was found in the wells installed as part of the exploration. A remediation district has been formed by Washoe County to effect a long-term cleanup of the groundwater for chlorinated solvent contamination.

Groundwater contamination will have an adverse effect on any potential dewatering operations, since current regulations would require the extracted groundwater to be treated before discharging either to the storm drain system or to the sanitary sewer. Past studies by others indicate groundwater in the Reno area may also contain high nitrate concentrations. The very strict discharge standards for the Truckee River precludes disposal of extracted groundwater to the river. Because of these severe environmental constraints, specialized construction techniques and permanent waterproofing of any subsurface corridor (e.g., trench or tunnel) is proposed, to minimize groundwater treatment issues.

The hydraulic conductivity of the soils below the groundwater surface was evaluated using several methods. Estimates of permeabilities were developed using empirical methods based on grain size distribution. Laboratory hydraulic conductivity testing was also performed on selected finer grained samples. Field hydraulic conductivity testing included slug tests in eleven wells and one constant pumping rate aquifer test during which six wells were monitored. Slug test hydraulic conductivity values varied greatly from location to location with coefficient of permeability k-values ranging from 2.5 feet to 1200 ft/day ($8.8 \times 10^{-4}$ to $4.2 \times 10^{-1}$ cm/sec). In the vicinity of the constant rate pumping test a hydraulic conductivity bulk value of 200 ft/day ($7.2 \times 10^{-2}$ cm/sec) was assessed. The results of the field pumping test suggest that highly permeable coarse-grained gravel layers dominate the groundwater flow. The pumping test revealed that initial inflow rates of up to 15 gallons per minute per square foot of excavation might be encountered in highly permeable gravel/cobble lenses.

Further study of the means and methods for constructing any of the alternatives is ongoing and will be required as part of the final design. However, preliminary evaluations of methods to build either of the below grade alternatives indicate that building the trainway “in the dry” below the groundwater surface is impractical. Construction methods such as deep mixing, sheet piles, permeation grouting and soil freezing are not recommended because of restrictions due to high groundwater flows, abundant cobbles and boulders, and/or the high fines content in the soil matrix. Alternate construction methods which appear more favorable for the local subsurface conditions include jet grouting, soldier piles and lagging, structural diaphragm slurry walls, soil nailing, or combinations of these methods. Further refinement of design and construction methods for each of the alternatives will be the subject of future reports. These wall and invert reports will be developed as the preliminary engineering preparation proceeds.

**GEOTECHNICAL ENGINEERING REPORT**
**PROPOSED RENO RAILROAD CORRIDOR EIS**
**RENO, NEVADA**

# 1. INTRODUCTION AND SCOPE

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## 1.1 Project Background/Description

This report presents the results of our geotechnical study for the proposed Reno Railroad Corridor. This report was prepared as part of Kleinfelder's contract to support the efforts of the Nolte Team's preparation of an Environmental Impact Statement. The overall project limits are from West 2nd Street (Alternative 2) or approximately 980 feet west of West 2nd Street (Alternative 3) on the west to Sutro Street on the east, and in proximity to the existing Union Pacific Railroad Tracks, which are located about a half block north of Commercial Row, between 2nd and 4th Streets. The site location is shown on the vicinity map (Plate 1, Appendix A).

This report will focus on mainline alternatives along the current rail alignment, and a parallel Commercial Row shoofly alignments through the downtown Reno area. The proposed corridor is approximately 2.25 miles to 2.5 miles in length, and located within the existing Union Pacific railroad right-of-way width of 54 feet. The mainline is a double track system. The shoofly will likely be a single track in the central third of the project area, with a double track layout on either end. A maximum trench depth of approximately 34 feet may result in final maximum cuts on the order of 36 feet to 39 feet depending on the type of invert system selected.

The EIS scoping process, completed in August, 1999, has thus far limited reasonable and feasible alternatives to those which are contained entirely within the existing Union Pacific right-of-way. In addition to the no-build alternative, three alternatives within this right-of-way are under consideration, as follows:

Alternatives 2 and 3. Depressed Trainway: These alternatives involve constructing a 2.25 mile (Alternate 2) and 2.5 mile (Alternate 3) depressed trainway through central Reno, extending from approximately Sutro Street on the east to 2nd Street (or possibly west of 2nd Street for

Alternative 3) on the west. A 1.0% to 1.2% descent and ascent grade is currently being considered, which would result in lowering the rail by approximately 30 feet to 34 feet at the deepest point. Eleven at-grade road crossings would be constructed or reconstructed over the depressed trainway (including a new bridge at Evans). A roadway underpass/train bridge would also be constructed at Sutro Street, approximately where the railroad tracks would return to their present grade. For Alternative 2, six of the planned bridges along the west end of the alignment (Keystone Avenue, Vine Street, Washington Street, Ralston Street, Arlington Avenue, and West Street) would be partially elevated, to provide the required vertical clearance for trains below. All of the north-south cross streets could be reconstructed as at-grade bridges without the need for overpass structures for Alternative 3.

Alternative 4. Trainway in Cover and Cut Tunnel: This alternative involves construction of a shallow double track tunnel with adjacent access road, extending approximately 3000 feet through the central portion of Reno with a total corridor length of 14,050 feet. A western grade of approximately 1.2% and eastern grade of approximately 0.93% are currently planned, although these grades may be adjusted as the design progresses. The cover and cut tunnel would be built in stages with the goal of maintaining rail traffic in the existing alignment along the downtown core. For this alternative, temporary shooflys would be constructed at either end of the project.

### 1.2 Purpose and Scope of Work

The purpose of this study is to evaluate the feasibility of the proposed alternatives with respect to the observed subsurface conditions, and to provide our geotechnical recommendations and opinions for the topics summarized below.

- Geologic and seismic setting;

- Summary of existing soil conditions within the defined corridor vicinity, based on a review of other work to date, and incorporating new information developed for the current study. Particular attention is to be paid to how these conditions are expected to affect the planned construction;

- Summary of existing groundwater conditions within the defined corridor vicinity, including depth to groundwater, flow direction and rate, and water quality issues, including presence and characterization of contaminants. This information is based on review of previous studies, and new data obtained by the present study, and will focus on construction related and long-term impacts;

- Lateral earth pressures and drainage recommendations for bridge abutments and the conventional shallow retaining structures likely in the east and west ends of the corridor;

- General foundation systems recommended for bridge support, including preliminary parameters for conventional shallow spread foundation design. The parameters discussed will include preliminary soil bearing values, minimum footing depth, resistance to lateral loads, and applicable seismic site coefficient for use in structural design;

- Preliminary recommendations for permanent cut and fill slopes;

- Preliminary evaluation of construction alternatives for the depressed trainway walls and invert (bottom) with particular attention to reducing/eliminating seepage of groundwater into the trench both during and after construction;

- Preliminary evaluation of various bracing systems for the depressed portions of the trainway; and

- Pavement sections for reconstructed portions of the adjacent roadways.

Our scope of services consisted of background review, site reconnaissance, field exploration, field testing, laboratory testing, engineering analyses, and preparation of this report. This report is intended to provide data obtained from our field and laboratory testing for the project, and generalized, preliminary design information. Attached technical memoranda further discuss the following issues: soil control and treatment; groundwater disposal and treatment; underpinning; pavement sections; and bridge foundations (see Appendices H, I, J and K).

# 2. FINDINGS OF PREVIOUS INVESTIGATIONS

## 2.1 Geotechnical Soils Information

This section addresses historical information previously developed along the existing railroad alignment, but does not include information developed during our current 1999/2000 subsurface exploration/laboratory testing, and engineering analyses. These current results are discussed in Section 5 of this report.

Numerous geotechnical groundwater and other reports and information were reviewed in the course of this study. These references are listed in Section 9. Our review indicates that subsurface conditions in the study area generally consist of a near surface layer of fill or native fine grained flood plain silts and sand, overlying very coarse grained granular materials of the Tahoe Outwash Formation. Typically, the outwash soils consisted of well stratified discontinuous layers of dense to very dense silty sand, clayey sand and gravel with cobbles and boulders. Previous reports indicate that boulders up to eight feet in diameter had been encountered in the outwash formation during previous construction projects within the downtown vicinity.

Fill material was reported in numerous subsurface explorations from the west to the east ends of the defined corridor (including Studio 3 Apartments, Fitzgerald's Casino Hotel Pedestrian Walkway, Eldorado Hotel South Side Expansion Project and City of Reno Hydrocarbon Study). Fill materials consisted of sandy clays, clayey sand, silty sand and silty gravel. The depth of fill varied from approximately 3.5 feet to 18 feet, with deeper fill present near 2nd Street and Commercial Row between Sierra and Center Streets in downtown Reno. It is possible that fill was present at other locations explored but no anomalies or construction debris was observed which would demarcate the site soils from native materials.

In our 25 years experience in Reno, we have found that most downtown buildings are constructed on conventional shallow spread foundations bearing on the Tahoe Outwash Formation. Many downtown buildings include a single underground level. A few structures are supported on drilled piers extending through surface fills, transferring building loads to the underlying gravel/cobble outwash deposits. In general, bearing values are moderate to high for foundations extending into the outwash formation, with low associated settlements.

## 2.2 Depth to Groundwater

Our review of depth to groundwater data focused on those data from wells within 300 feet to 400 feet of the present rail alignment. This maximum distance was selected based on the apparent relatively steep gradient of the potentiometric surface (water table). Thus data from wells located further away than 400 feet may not be representative of site conditions.

Groundwater elevations encountered in the referenced reports for the central part of the study corridor varied from 23 feet to 36 feet below the existing ground surface. The highest groundwater levels recorded in referenced documents occurred in December 1982, when groundwater elevations within one block of the alignment varied from approximately 4459 feet to 4474 feet, corresponding to depths of 23 feet to 36 feet below existing grade. It is our understanding that Harrah's Casino, located on 2nd Street at Virginia Street, conducts on-going dewatering of their basement sumps. This dewatering operation creates a cone of depression of approximately 10 feet near the corner of 2nd and Center Streets. This cone of depression results in drawdown of approximately 1 foot to 3 feet along the existing rail corridor, between Center Street and Virginia Street. According to the geotechnical investigation report by SHB AGRA in July 1994, excavations for the expansion of the Eldorado Casino-Hotel encountered groundwater at 31 feet below existing grade (approximate elevation 4470 feet). Borings for the Wells Avenue Improvements in 1963, encountered groundwater at approximately elevation 4463 feet (27 feet below grade).

We contacted Washoe County to supplement the referenced information regarding historical and seasonal variations in groundwater levels. Currently, there is no known comprehensive program to monitor groundwater levels in downtown Reno, and so no continuous records are available. The Reno Folio includes a Hydrologic Map showing depth to groundwater from wells shallower than 150 feet in depth. This map, reproduced on Plate 1, Appendix G, shows groundwater depths ranging from 20 feet to just less than 40 feet in the project area. The text accompanying the map indicates the Truckee River water levels fluctuate 6 feet to 8 feet as a maximum seasonal variation, but does not directly comment on seasonal groundwater level fluctuations. Our research (J.H. Kleinfelder & Associates, 1982; SRK, 1995; Westec and SRK, 1994) revealed a reasonably long record of information on depth to water table for one boring located approximately 125 feet south of the existing rails, in Center Street. At this boring, readings taken in December 1982, July 1993, June 1995, and April 1998 only varied by 2 feet, with depths to groundwater varying from 22 feet to 24 feet below grade with the highest level recorded in April, 1998. In general, the highest groundwater elevations are typical of the late spring and early summer, with the lowest levels in midwinter.

## 2.3 Groundwater Quality and Flow

In general, groundwater quality in the Reno area is considered to be "good" and meets or exceeds present drinking water standards except for the presence of manmade chemicals.

Discharge of any construction generated water to the Truckee River would need to meet stringent discharge standards. These standards apply to not just the man-made organic chemicals found in the groundwater but also to naturally occurring or seeming benign chemicals that have been concentrated in the groundwater due to man's past activities. Some of these chemicals and parameters include nitrogen, phosphorous and total dissolved solids. These chemicals are of specific concern since the facilities that discharge to the river already face difficulties in meeting the standards for these parameters. For instance, the maximum allowable concentration of total dissolved solids allowed in a discharge is 120 mg/l. However, the Nevada drinking water standard for total dissolved solids is 500 mg/l. In addition to a maximum concentration for these and other constituents, there is a "Total Maximum Daily Load." This load is assessed per constituent and is based on the needs of the aquatic ecosystem. The standards are assessed in pounds per day and take into account both concentration and flow rates. Therefore, a treatment system for the discharge to the river would need to be designed so as to meet both the concentration and loading requirements on the Truckee River.

Multiple previous studies have reported on groundwater quality in the downtown Reno area. Sierra Pacific Power Company (SPPCo) conducted a study and presented results in January 1989. Tetrachloroethene (PCE) was found to be present in samples from 7 of 12 existing monitoring wells at concentrations ranging from 2.5 parts per billion (ppb) to 70 ppb. Westec and SRK performed an investigation, dated March 1994, of downtown groundwater quality. Of 21 wells sampled, 10 contained PCE in concentrations ranging from <10 ppb to 480 ppb. The 10 wells containing PCE are located in an area approximately bounded by Keystone Avenue to the west, the Nevada Experimental Farm to the north, Pine Street to the south, and Wells Avenue to the east. SRK performed sampling of 15 existing wells in 1993 and 1995 to assess the presence of benzene, toluene, ethylbenzene, xylene (BTEX), and free phase diesel floating product. Results were presented in a report dated July 13, 1995. Non-detect concentrations for BTEX were reported for samples from all 15 wells. In 1993, 3 of 15 wells had floating product ranging in thickness from 0.36 inches to 4.92 inches. In 1995, the same three wells were sampled and two exhibited a sheen and the third had less than 0.125 inch of floating product. The three wells with reported free product are located along 2nd Street from Virginia to Sierra Streets.

Generally, PCE has been found in the groundwater in the downtown Reno area from Keystone Avenue (west) to Wells Avenue (east) and the Experimental Farm (north) to Pine Street (south); our current study area for the Railroad Corridor Project lies within these boundaries. Concentrations of PCE in groundwater are highly variable and indicate multiple release sources in the downtown area. Hydrocarbon products have also been found to be present in localized areas in the downtown Reno area, specifically near the intersections of 2nd Street with Virginia and Sierra Streets.

Cohen and Loeltz (1964) describe the movement of groundwater in the Reno area. The groundwater flow direction is predominantly west to east, parallel to the Truckee River. Groundwater flow direction assessments provided in Westec/SRK (1994) and SRK (1995) reports also show the general direction of groundwater flow as eastward in the vicinity of the project area. The hydraulic gradient of the uppermost portion of the aquifer is about $6.6 \times 10^{-3}$ based on groundwater flow maps in these reports. Localized dewatering activities, such as Harrah's dewatering, can cause localized depth to groundwater and ground flow direction variations. According to NDEP, dewatering pumpage in Harrah's basement averages 132 gallons per minute (190,000 gallons per day), resulting in approximately 10 feet of drawdown. This pumping results in 1 foot to 3 feet of drawdown along the railroad alignment at its Center Street crossing. Drinking water production wells near downtown Reno generally produce water from deep confined or semi-confined alluvium. These production wells have little or no direct impact on the near-surface aquifer, and are therefore not expected to impact the planned Railroad Corridor maintenance construction.

# 3. GEOLOGIC AND SEISMIC SETTING

## 3.1 Geologic Setting

The site is located in a transition area between the Sierra Nevada geologic province to the west and the Basin and Range geologic province to the east. The Sierra Nevada mountains were formed by large intrusions of molten granitic rock during Mesozoic time. Subsequent faulting during Tertiary time raised the mountain range to its present position. The Basin and Range Province was formed by numerous north-south trending normal faults which formed the horst and graben morphology of most of Nevada. Most of the displacement on these faults occurred during Tertiary time, although earthquake activity continues to the present in much of the Basin and Range Province. The mountain ranges in western Nevada are primarily composed of Mesozoic or Early Tertiary intrusive and Tertiary volcanic rocks. The intervening basins, such as the Truckee Meadows basin, consist of deep accumulations of Early Cenozoic to Quaternary age sediments.

Repeated Pleistocene age glaciations followed in the higher ranges west of the site which resulted in glacial outwash, debris flows, and flood materials being deposited in the Truckee Meadows area. Recent unconsolidated deposits occur along present day drainage channels as the result of stream outwash carried down from the Carson Range and Truckee River Basin.

The project site is located in the northwestern portion of the Truckee Meadows basin. The Geologic Map of the Reno Folio (1976) shows the project site is located in an area of Tahoe stage glacial outwash which consists of sandy gravel, cobbles and boulders and interbedded coarse sand. The cobbles and boulders are well rounded, predominantly andesite rock with some granitic rock and range up to 8 feet in diameter, according to construction/earthwork reports. These gravel deposits extend to depths of more than 100 feet. The Tahoe age glacial outwash deposits have a relatively well developed soil profile consisting of clayey sand with a thin surface layer of silty sand. This unit is believed to have been deposited during catastrophic flooding of the Truckee River during glacial periods. Bierkland (1968) describes the process of ice damming of Lake Tahoe during glacial intervals, followed periodically by ice breaking, flooding, and depositing of sand, silt, gravel, and some large boulders and frequent cobbles in very high energy flows. Locally, man made fill is present at the surface throughout the downtown area.

## 3.2 Faulting and Seismicity

The railroad corridor and the entire Reno - Carson City Urban Corridor is dominated by the faults of the Basin and Range geomorphic province. The tectonic setting of the Basin and Range province is characterized by active extension expressed by evenly spaced, sub-parallel mountain ranges and intervening alluviated basins. Basins and ranges throughout much of the province are bounded by north/south trending normal faults. The study area is considered seismically active because of the presence of numerous active or potentially active faults. The tectonics in the area is mainly influenced by the Sierra Nevada Frontal fault system (also known as Carson Range fault system). The Sierra Nevada Frontal fault system consists of a series of north-south trending fault zones generally thought to extend from Reno to south of Woodfords, California. It is one of the largest and most active fault systems in Nevada. This system consists of Mount Rose fault zone, the Carson City fault and the Genoa fault zone.

In the vicinity of the railroad corridor, the Mount Rose fault zone of the Carson Range fault system, about 2-1/2 miles to the southeast, and the Northwest Reno fault zone, about 2 miles to the west, are the closest active faults. These active faults are believed to have experienced offset within the past 10,000 years. The project site is not within any part of mapped Earthquake Fault Zones and no active shear zones are known to exist at the site. However, the site will experience strong ground shaking in case of a seismic event at one of the nearby faults. Other active and potentially active faults which can have significant impact on the site are listed in the Table 1, located at the end of this section of the report. The fault parameters presented in Table 1 are primarily based on data provided by dePolo et al. (1997). Additional information has been obtained from Frankel et al. (1996).

According to California Division of Mines and Geology (CDMG), an active fault is defined as a fault that has produced surface displacement within Holocene time (approximately the past 11,000 years). Based on this, we have considered all of the faults showing evidence of Holocene surface displacement as active and the remaining faults as potentially active. However, it should be noted that different regulatory agencies use different criteria to establish fault activity.

The ground motions in terms of peak ground accelerations at the site were estimated using the deterministic method. The deterministic method utilizes the maximum earthquake magnitude associated with the faults considered in the analysis. This method does not account for the relative activity of the fault and assumes that the site will be subjected to the ground shaking from a maximum earthquake associated with that fault. Many attenuation relationships have been developed to estimate the variation of peak ground surface acceleration with earthquake magnitude and distance from the site to the source of an earthquake. Of these relationships, we have selected relationship presented by Boore et al. (1994,1997) because of its wide acceptance by seismologists. This relationship has also been used in developing recent National Seismic Hazard Maps (Frankel et al., 1996) for the State of California and Nevada. This relationship uses an estimate of site shear wave velocity in the analyses. Therefore, an average site shear wave velocity of 310 m/s, recommended by the authors for typical soil deposits, was used in our analyses. Table 1 shows the mean and mean + 1 standard deviation values of peak ground accelerations at the site due to the maximum earthquake associated with each fault.

Historically, the area has been subject to intense seismic activity. Some of the magnitude 6.0 or greater nearby events include the 1857 (Magnitude 6.2 [M6.2] Richter Scale) Western Nevada earthquake, the 1860 (M7) Pyramid Lake area earthquake, the 1868 (M6) and 1869 (M6.7) Virginia Range earthquakes, the 1869 (M6.1) southern Virginia Range earthquake, the 1887 (M6.3) Carson Valley earthquake, the 1914 (M6) Verdi area earthquake, the 1914 (M6.4) northern Virginia Range earthquake, the 1933 (M6) northern Singatze Range earthquake, the 1948 (M6) Verdi area earthquake, the 1966 (M6) Truckee earthquake, and the 1994 (M6) Double Spring Flat earthquake.

According to dePolo et al. (1997), within the Reno-Carson City Urban corridor, the probability of at least one magnitude 6 event over a 50-year period is between 34% and 98%, whereas, the probability of a magnitude 7 event is between 4% and 50%. According to Earthquake Scenario for Western Nevada (1996), for a magnitude 7.1 earthquake on the Carson Range fault zone, several important transportation routes within the corridor will be affected by surface faulting, liquefaction, settlement, rockfalls and landslides, fallen debris, bridge and overpass damage, and bridge abutment settlement. It is not likely that any of the geologic hazards (rupture, liquefaction, rockfalls or landslides) will occur within the Railroad Corridor. The potential for damage to future bridges can be reduced by providing proper design and construction techniques (See Section 6.3 of this report).

**TABLE 1**

SIGNIFICANT FAULTS AND THEIR PARAMETERS

| Fault Name | Closest Distance to Site (km) | Magnitude of Maximum Earthquake * | Slip Rate (mm/yr) | No. of Events /Year (x $10^{-5}$) | Deterministic Peak Ground Acceleration (g) | |
|---|---|---|---|---|---|---|
| | | | | | mean | mean+1σ |
| Northwest Reno** | 3 | 6.6 | 0.01-0.08 | 2.5-20 | 0.45 | 0.75 |
| Mount Rose | 4 | 7.1 | 0.1-0.5 | 8.2-41 | 0.55 | 0.92 |
| Eastern Reno Basin** | 7 | 6.9 | 0.01-0.1 | 1.3-13 | 0.40 | 0.67 |
| Spanish Springs Valley | 7 | 6.9 | 0.05-0.3 | 6.4-39 | 0.40 | 0.67 |
| Peavine Mountain | 9 | 7.0 | 0.05-0.3 | 2.1-74 | 0.37 | 0.62 |
| Freds Mountain | 9 | 7.0 | 0.05-0.2 | 5.1-21 | 0.37 | 0.62 |
| Spanish Springs Peak | 11 | 6.6 | 0.05-0.3 | 13-74 | 0.27 | 0.45 |
| Northern Virginia Range** | 12 | 6.6 | 0.01-0.1 | 2.5-25 | 0.25 | 0.42 |
| Little Valley ** | 15 | 6.9 | 0.05-0.3 | 6.4-39 | 0.25 | 0.43 |
| Hungry Valley** | 17 | 6.6 | 0.001-0.1 | 0.25-25 | 0.20 | 0.34 |
| Granite Hills | 18 | 6.6 | 0.05-0.3 | 13-74 | 0.19 | 0.32 |
| Peterson Mountain | 20 | 7.0 | 0.05-0.3 | 5.1-31 | 0.22 | 0.37 |
| Olinghouse** | 20 | 7.1 | 0.05-0.3 | 4.1-25 | 0.23 | 0.39 |
| Comstock** | 23 | 6.6 | .005-.05 | 1-13 | 0.16 | 0.27 |
| W. Warms Springs Valley | 25 | 6.9 | 0.005-0.1 | 0.64-13 | 0.18 | 0.30 |
| Incline Village** | 31 | 6.6 | 0.01-0.1 | 2.5-60 | 0.13 | 0.22 |
| East Tahoe | 37 | 7.0 | 0.1-0.5 | 10-51 | 0.14 | 0.23 |
| Carson City | 38 | 6.8 | .005-.05 | 8-48 | 0.12 | 0.21 |

* moment magnitude

** potentially active faults (all other faults are considered active)

We performed a seismic hazard analysis with respect to peak ground accelerations for a Design Basis Earthquake (DBE) for the Railroad Corridor project in Reno, Nevada. According to 1997

Uniform Building Code (UBC), DBE is defined as a ground motion having 10% probability of exceedance in 50 years (return period of approximately 475 years).

In general, the materials underlying the site are considered alluvial for the seismic risk analyses. Out of several available attenuation relationships to assess peak ground accelerations, we have selected the relationship proposed by Boore et al. (1994, 1997) which uses the average site shear wave velocities in the analysis. This relationship has also been used in developing recent National Seismic Hazard Maps (Frankel et al., 1996) for the State of California and Nevada. This predictive relationship was developed from statistical analyses of recorded earthquakes from Western North America, including the records from the 1989 Loma Prieta earthquake and 1992 Landers earthquake. The attenuation relationships provide mean values of ground motions associated with one set of parameters: magnitude, distance, site soil conditions, and mechanism of faulting. The uncertainty in the predicted ground motion is taken into consideration by including a magnitude dependent standard error in the probabilistic analysis. We have performed our analyses assuming the shear wave velocity at the site of about 310 m/s for the top 30 m (100 feet), as suggested by Boore et. Al. (1997) for a typical alluvial soil site.

Probabilistic modeling procedure was used to estimate the peak ground motions corresponding to the design level earthquake. The probabilistic analysis approach is based on the characteristics of the earthquake and of the causative fault associated with the earthquake. These characteristics include such items as magnitude of the earthquake, distance from the site to the causative fault, maximum credible earthquake, length, and activity of the fault. The effects of site soil conditions and mechanism of faulting are accounted for in the attenuation relationship for this site.

The theory behind seismic risk analysis has been developed over many years (Cornell, 1968, 1971; Merz and Cornell, 1973) and is based on the "total probability theorem" and on the assumption that earthquakes are events that are independent of time and space from one another. According to this approach, the probability of exceeding $P_E(Z)$ at a given level of ground motion, Z, at the site within a specified time period, T, is given by

$$P_E(Z) = 1 - e^{\vartheta(Z)T}$$

where $\vartheta(Z)$ is the mean annual rate of exceedance of ground motion level Z. Different probabilities of exceedance may be selected, depending on the level of performance required.

We have performed our risk analysis assuming an average Vs of 310 m/s in the top 30m (100 feet) for the proposed site. Based on the result of our seismic risk analyses, the calculated peak horizontal ground surface acceleration (in units of gravity) for the DBE together with the return period and annual probability of occurrence is presented in the Table 1A.

**TABLE 1A**

PEAK GROUND ACCELERATION

| Earthquake Level | Return Period (years) | Annual Probability of Exceedance | Probability of Occurrence | Peak Horizontal Acceleration |
|---|---|---|---|---|
| DBE | 475 | 0.0021 | 10% in 50 years | 0.42g |

# 4. METHODS OF STUDY

## 4.1 Field Exploration and Testing

### 4.1.1 Main Alignment Drilling

Our field exploration was performed in accordance with the Reno Railroad Corridor EIS/PE Scope of Services, Task 3, Section 3.b.1 dated June 14, 1999. The subsurface exploration for the preferred alternative (main alignment) consisted of drilling 36 borings, as follows: three deep borings to depths of 119.5 feet to 125 feet; 20 shallow borings to depths of 2 feet to 82 feet; 10 bridge borings to depths of 39 feet to 62.5 feet and three large 39-inch diameter borings to depths of 55 feet. Our selection of boring locations was based on the anticipated project alignment, utility clearance, and site access. Prior to our subsurface exploration, Underground Service Alert and Spectrum-Gasch Geophysics of Los Angles, California were contacted to clear underground utilities at each boring location. In areas of existing pavements, the asphalt section was cored by Accurate Drilling and Sawing, LLC of Reno, Nevada. Measured pavement sections are described on the pertinent boring logs in Appendix A.

Locations of the borings shown on the Site Plans (Plates 2 through 4, Appendix A) were approximated by measuring from features shown on the site plan using a wheel odometer. Elevations shown on the boring logs were obtained by interpolation between contour lines shown on the site plan, which was provided by Stantec, Inc. of Reno, Nevada. These locations and elevations should be considered accurate only to the degree implied by the method used. It should be noted that there are no borings B5, B13 and B31. These borings were deleted to add pump test borings P1, P2 and P3 located near the deepest section of the fully depressed trainway alternative.

The borings were advanced using four drilling methods. Two borings (B-6 and B-33) were advanced using a truck mounted CME55 drill rig with continuous flight, hollow stem augers provided by Andresen Exploration Drilling of Reno, Nevada. The borings B-6 and B-33 met practical refusal in near surface cobbles at depths of 12 feet and 2 feet, respectively.

Eighteen borings were advanced using the ODEX/Stradex drilling method, creating a 6-inch diameter borehole. This method uses an eccentric bit on a down-hole percussion hammer. The lower portion of the bit chops and rotates while the upper portion of the bit moves outward to enlarge the boring to allow casing to follow the bit. Seventeen of the ODEX/Stradex borings were advanced using a Schramm T-64 Drill Rig provided by Spectrum Exploration, Inc. of Stockton, California. Air pressure to the percussion hammer on the Spectrum drill rig was monitored during drilling. The purpose of the monitoring was to help identify any anomalies, such as extremely dense materials which would require higher drilling forces. The recorded air pressures are presented on the Spectrum ODEX/Stradex boring logs in Appendix A. The final ODEX/Stradex boring (P2) was drilled using a Speedstar 70K drill rig provided by Sargent Irrigation of Reno, Nevada. This second drill rig was used to advance a 12-inch diameter borehole for the installation of a downhole pump for the collection of water infiltration data.

Thirteen borings were advanced using a SONIC drill rig provided by Alliance Environmental, Inc. of Phoenix, Arizona. The SONIC method using a specially designed hydraulically powered drill head or oscillator. The oscillator generates adjustable high frequency vibration forces, which are transferred down the steel casing and drill pipe to the face of the bit. When the vibrations in the drill rod and bit coincide with the natural frequency of the subsurface materials resonance occurs and the boring advances through soil or rock by displacement, shearing and fracturing. The drill rig advances a 4-inch diameter core barrel for continuous sampling, providing a nearly complete record of subsurface stratigraphy.

Three large diameter borings were advanced by Condon-Johnson & Associates, Inc. of Oakland, California using a Soil MEC R722HD auger drill rig. The borings had an average diameter of 39 inches (1 meter) and were advanced to a depth of 55 feet. Three types of drill bits were used, an auger bit for general drilling, a core barrel for large boulders and dense gravel and cobbles, and a bucket to remove cuttings or debris from the excavation. The purpose of the large borings was to help evaluate the amount of oversized boulders, the type of matrix material and how these conditions could affect construction methods such as deep mixing method (DMM).

A field engineer or geologist logged the soil conditions exposed in the borings and collected bulk and relatively undisturbed samples for laboratory testing. Soil conditions encountered are presented on the boring logs included as Plates 5 through 40 in Appendix A. A key to boring logs and description of the Unified Soil Classification System used to identify the site soils are also provided in Appendix A (Plates 53 and 54). Photographs of samples obtained from the SONIC drilling method are presented on Pages B-1 through B-16 in Appendix B.

Relatively undisturbed soil samples were obtained while drilling with the ODEX/Stradex and hollow stem auger drill rigs by driving a 2-inch inside diameter, 2-1/2-inch outside diameter Modified California Sampler, containing thin brass liners, into the bottom of the boring. The number of blows required to drive the last 12 inches of an 18-inch drive with a 140 pound hammer dropping 30 inches is recorded as the blows per foot (Blow Count) on the boring logs. When the sampler was withdrawn from the boring, the brass liners containing the samples were removed, examined for logging, labeled and sealed to preserve the natural moisture content for laboratory testing.

At select locations, our engineer or geologist also performed undrained shear strength measurements on fine grained soil samples using the pocket penetrometer device. The results of this testing are presented on the boring logs.

After borings were completed, they were backfilled with grout in compliance with the State of Nevada Department of Conservation and Natural Resources requirement, NAC 534.4371, dated January, 1998. Twelve of the borings were finished as monitoring wells or as a pumping well (P2), although boring B-32 was found to be dry when we attempted to develop/sample it. Well design information is provided on the boring logs in Appendix A.

#### 4.1.2 Shoofly Drilling

The proposed shoofly as shown on the Union Pacific Railroad Aerial Diagram, dated February 24, 1997 was investigated by drilling twelve shallow borings along the alignment using a truck mounted CME55 drill rig with continuous flight, hollow stem augers. The drill rig was provided by Andresen Exploration Drilling of Reno, Nevada. The borings met practical refusal in near surface cobbles and boulders in all twelve borings at depths of one to seven feet. However, since only minor cuts and fills are anticipated in most areas, this shallow refusal did not impair the integrity of the shoofly exploration program, and adequate information was developed for preliminary engineering purposes.

Locations of the borings shown on the Site Plan (Plates 2 through 4, Appendix A) were approximated by measuring from features shown on the site plan using a wheel odometer. These locations should be considered accurate only to the degree implied by the method used.

A field engineer logged the soil conditions exposed in the borings and collected bulk and relatively undisturbed driven samples for laboratory testing. Soil samples were obtained while drilling by driving a 2-inch inside diameter, 2-1/2-inch outside diameter Modified California Sampler, containing thin brass liners, into the bottom of the boring. As described above, the number of blows required to drive the sampler is recorded as the blows per foot (Blow Count) on the shoofly boring logs. Soil conditions encountered are presented on the boring logs included as Plates 41 through 52 in Appendix A.

### 4.1.3 Sampling for Contaminant Testing

*Soil*

Soil samples were collected on 10 foot intervals (or as geologic conditions allowed) for the upper forty feet of the borings. Four soil samples per boring were field screened for hydrocarbons using a PetroFlag™ test kit. One sample from each boring having the highest hydrocarbon concentrations, as indicated by PetroFlag results, was submitted to Alpha Analytical of Sparks, Nevada (Alpha) for total petroleum hydrocarbons and volatile organic compounds. One soil sample per boring for the proposed temporary shoofly track was field analyzed for hydrocarbons using the PetroFlag test kit. Results are shown in Table 1, Appendix D. The PetroFlag test kit was used per the operating instruction located in Appendix D.

Soil samples were collected in the field using several different methods that were dependant on the type of drilling being performed. Samples collected during drilling using the ODEX/Stratex method were collected in new brass tubes placed in the sampler. Samples collected from borings installed using the SONIC drilling system were obtained from within the plastic (polyethylene) core sleeve. During the installation of the large diameter borings (LG-1 through LG-3), soil samples were collected directly off the large diameter augers. Direct sampling of cuttings from the auger also occurred during the installation of the borings along the proposed shoofly alignment. All the samples not collected in brass tubes were placed in laboratory supplied containers.

Once the samples were containerized they were placed in a cooler with ice. Those samples transmitted to a laboratory for analysis were submitted under a state of practice chain-of-custody protocol, within their respective U.S. EPA established holding times.

*Groundwater*

Groundwater samples were collected in the upper twenty feet of the first aquifer encountered from 11 borings which were finished as monitor wells and which had water in them. These samples were collected from within the drill stem using a new disposable dedicated bailer for each boring. Groundwater samples were collected from these wells following well development and purging. Well development consisted of swabbing and pumping each well to remove fine grained sediment from the well and sand pack. Approximately 100 gallons of sediment and development water were removed from each monitoring well. Approximately 24 hours after development of the wells, they were purged of three to five well volumes, prior to sampling. During purging the temperature, specific conductance and pH of the water were monitored. These data are presented in Table 2, Appendix D. Each sample was labeled and placed in a cooler with ice. These samples were transmitted to Alpha for analysis of volatile organic (VOC) using EPA Method 601 and for Total Petroleum Hydrocarbons (TPH) by EPA Method 8015.

### 4.1.4 Field Slug and Pumping Testing

*Slug Tests*

Slug tests were performed in 11 borings that were completed as monitor wells. These tests were performed to assess the hydraulic conductivity of the uppermost portion of the aquifer within the immediate area of the boreholes. A Hermit 3000 data logger and pressure transducer were utilized in testing. A displacement slug constructed of a 3 foot to 5 foot length of PVC pipe was used to rapidly displace a known volume of water in the well. A 1-inch diameter slug was used in the 2-inch diameter wells, and a 3-inch slug was used in the 6-inch diameter well (P-2).

The "slug out" method was used for all slug tests. Each test was performed by first measuring the static water level followed by installation of a pressure transducer and the displacement slug below static water level. When the water level in the well had recovered to a static condition the slug was quickly removed. The resulting drawdown and recovery of the water level in the well was recorded by the data logger at a rate of at least one data point per second.

Test data were analyzed using the solution method of Bouwer and Rice, 1976. This solution was chosen since it accounts for variables such as the degree of partial penetration of the well and other well design factors. In addition, it was developed for unconfined aquifer conditions such as those encountered at the project site. Plots of water level versus time with linear fits and calculations are shown in Appendix E. Slug test analysis results are summarized in Table 2.

## TABLE 2

MONITORING WELL SLUG TESTING RESULTS

| Well Number | Hydraulic Conductivity[1] (feet/day) | Hydraulic Conductivity[1] (cm/sec) | Transmissivity[2] (feet²/day) | Lithology |
|---|---|---|---|---|
| B-14 | 510 | $1.8 \times 10^{-1}$ | 9,900 | Gravelly sand with silt |
| B-19 | 2.5 | $8.8 \times 10^{-4}$ | 78 | Gravelly sand trace silt trace cobbles (35′-45′)<br>Sandy gravel trace silt (45′-55′) |
| B-20 | 260 | $9.2 \times 10^{-2}$ | 6,400 | Clayey Gravel with sand (26′-28′)<br>Sandy gravel with cobbles, boulders (28′-36′)<br>Clayey gravel with sand (36′-40′) |
| B-22 | 250 | $8.8 \times 10^{-2}$ | 6,300 | Clayey sandy gravel with cobbles |
| B-24 | 620 | $2.2 \times 10^{-1}$ | 17,000 | Sandy gravel with cobbles |
| B-27 | 390 | $1.4 \times 10^{-1}$ | 11,000 | Silty sand (25′-29′)<br>Gravelly sand trace silt (29′-32′)<br>Clayey sand (32′-34′)<br>Silty clay(34′-38′) |
| B-28 | 490 | $1.7 \times 10^{-1}$ | 9,900 | Silty sand |
| B-29 | 280 | $9.9 \times 10^{-2}$ | 4,700 | Silty sandy gravel trace clay (33′-35′)<br>Sandy clayey gravel (35′-40′) |
| P-1 | 180 | $6.4 \times 10^{-2}$ | 3,600 | Silty gravel with sand trace clay |
| P-2 | 1,200 | $4.2 \times 10^{-1}$ | 36,000 | Sandy gravel with cobbles (17′-24′)<br>Sandy gravel with some clay (24′-39′)<br>Sandy gravel with clay (39′-50′) |
| P-3 | 410 | $1.4 \times 10^{-1}$ | 10,000 | Sand trace silt (26′-31′)<br>Clayey sandy gravel (31′-36′)<br>Clayey sand (36′-40′) |

Notes: 1 Hydraulic conductivity assessment based on monitor well partial penetration into an aquifer assumed to have a thickness of 30 feet, corresponding to an aquifer base at approximately 50 feet below grade.

2 Transmissivity assessment based on monitor well partial penetration into aquifer assumed to have a base at 50 feet below grade.

*Aquifer Pumping Test*

Kleinfelder performed a 16 hour constant rate pumping test on September 21-22, 1999. Well P-2 was used as the pumping well; it is located near the Sierra Pacific Power Company (SPPCo) substation west of Wells Avenue. This location was selected for the pump test location for three reasons: the proposed trench is in once of its deepest cuts here; there is adequate open area to drill monitor wells near the pumping well; and two monitor wells already exist in the area, owned by the Nevada Department of Environmental Protection (NDEP). Drawdown data was recorded using a pressure transducer and a Hermit 3000 data logger. The drawdown graphs are presented in Appendix F. These data suggest that the water level in the well reached near steady-state conditions within five minutes of pumping at this rate.

Sargent Irrigation installed a twelve stage 40 horsepower submersible pump in the well for the continuous pumping test in order to increase pumping capacity. The pump intakes were at a depth of 38 feet in the well, 16 feet below the water table (static water level was approximately 22 feet below the ground surface [bgs]). Pumping of the well began at 11:04 AM on September 21, 1999, and continued for 16 hours to end at 3:06 AM on September 22, 1999. Discharge rate was monitored using an inline mechanical flow meter. Discharge water was pumped through a pair (in parallel) of canisters each containing 1,000 pounds of granular activated carbon. The carbon canisters were PV 80 models provided by U.S. Filter, West States. After treatment the water was discharged through a manhole into a City of Reno inverted siphon on the sanitary sewer. This siphon is located at the northwest intersection of the Truckee River and 2nd Street. Discharge to the sewer was performed under permit issued by the City of Reno. Monitoring of petroleum and volatile organic compounds identified in previously collected samples from the local wells was performed per permit requirements during the test. Table 4 in Appendix D presents the analytical data from these monitoring activities.

Six wells located within 400 feet of the pumping well were monitored during the test: P1, P2, P3, B27, 7NS, and 8ND. Wells 7NS and 8ND are owned by the Nevada Division of Environmental Protection (NDEP), and were installed under the oversight of Westec/SRK per their report dated 1994. Well 7NS is screened in the uppermost portion of the aquifer, 13.7 feet to 23.7 feet below ground surface and Well 8ND is screened in a lower portion of the aquifer (176 feet to 186 feet bgs). These wells are referred to in this report as WCS and WCD, respectively. Prior to the test Kleinfelder gained approval to obtain water level data from these wells from NDEP.

Static water levels were measured prior to the test. Dynamic drawdown water level data were measured during the pumping phase of the test. Water level recovery data were collected for eight hours following test termination. Water level data was recorded to the nearest 0.01 foot using either an electronic tape or a pressure transducer and Hermit 3000 data logger. The elevations of the measuring points were surveyed for these wells. In addition, a point on the Truckee River was also surveyed. These data were used to prepare groundwater gradient maps presented in Appendix G.

The pumping portion of the test operated for a total of 16 hours 2 minutes (962 minutes). The initial pumping rate was 148 gpm. The water level in the pumped well rapidly declined and reached the pump intake level in approximately 10 minutes. After reaching the pump intake elevation the pumping test was transformed into a constant drawdown test with a drawdown of 16 feet in the well bore. Pumping rates slowly declined to 60 gpm at the end of the test. The time weighted average pumping rate during the test was 95 gpm. The average specific capacity of the well during the test was 5.9 gpm/ft; at the end of the test the specific capacity was 3.8 gpm/ft. The test was terminated at 3:06 AM on September 22 because of pump failure caused by cavitation in the pump bowls and heat build up in the motor. The decline in flow rate to the well may have resulted from a laterally discontinuous production zone that caused a “discharge–type” hydraulic boundary condition.

Preliminary analysis of pumping test data was performed using “ESI Aquifer$^{Win32}$” software by the Theis (1935), Cooper and Jacob (1946), and Neuman (1972) drawdown solutions, and by the Theis recovery method. These solution methods were chosen based on the apparent water table conditions under which the shallow aquifer exists. The foundation thickness used in calculating transmissivity is the screened interval of the well, as shown on the Boring Logs in Appendix A. The test data were also evaluated by application of the U.S. Geological Survey MODFLOW model to accommodate the variable withdrawal rate during the test. These results are summarized in Table 3.

## TABLE 3

AQUIFER PERFORMANCE TEST RESULTS

| Well Number | Analytical Method & Data Used | Hydraulic Conductivity[1] (ft/day) | Hydraulic Conductivity[1] (cm/sec) | Trans-missivity ($ft^2$/day) | Specific Yield ($ft^3/ft^2/ft$) | Notes |
|---|---|---|---|---|---|---|
| P-1 | Theis, 1935 Drawdown Data | 320 | 1.1x10$^{-1}$ | 9,500 | 0.071 | |
| P-3 | | 180 | 6.4x10$^{-2}$ | 5,300 | 0.072 | |
| B27 | | 210 | 7.4x10$^{-2}$ | 6,200 | 0.061 | |
| MW WCS | | 250 | 8.8x10$^{-2}$ | 7,500 | 0.11 | Middle time data |
| MW WCS | | 100 | 3.5x10$^{-2}$ | 3,000 | 0.090 | Late time data |
| P1, P3, B27 & WCS grouped | | 370 | 1.3x10$^{-1}$ | 11,000 | 0.090 | Time adjusted by radius squared |
| P-1 | Cooper & Jacob, 1946 Drawdown Data | 690 | 2.4x10$^{-1}$ | 20,600 | 0.053 | |
| P-3 | | 450 | 1.6x10$^{-1}$ | 13,500 | 0.056 | |
| B27 | | 500 | 1.8x10$^{-1}$ | 15,000 | 0.044 | |
| MW WCS | | 330 | 1.2x10$^{-1}$ | 10,000 | 0.073 | |
| Pumped Well P-2 | Theis, Recovery Data | 130 | 1.7x10$^{-1}$ | 4,000 | - | Late time data |
| P1, P3, & WCS grouped | USGS MODFLOW Drawdown Data | 200 | 7.1x10$^{-2}$ | 6,100 | 0.070 | Calibrated to observed drawdowns at the end of test; used variable pumping rate schedule. |

Notes: 1 Hydraulic conductivity calculated from transmissivity assessment and an assumed wetted aquifer thickness of 30 feet.

## 4.2 Laboratory Testing

### 4.2.1 Geotechnical Laboratory Testing

Geotechnical laboratory testing for the project included tests for evaluating both index and engineering properties of soils. Index tests included soil moisture content, unit weight, soil particle gradation, and plasticity characteristics. Tests for engineering properties included soil strength, compressibility, and subgrade support characteristics. We performed the following laboratory tests on selected soil samples:

- Particle Size Analysis (ASTM D422)
- Percentage of Soils Finer than the No. 200 Sieve (ASTM D1140)
- Liquid Limit, Plastic Limit and Plasticity Index (ASTM D4318)
- Unit Weight by the Drive-Cylinder Method (ASTM D2937)
- Moisture Content of Soil (ASTM D2216)
- One-Dimensional Consolidation (ASTM D2435)
- Direct Shear Strength Under Consolidated Drained Conditions (ASTM D3080)
- R-Value (Nevada Test Method T115)
- Constant Head Permeability (ASTM D2434)
- Consolidated Undrained Triaxial Compression (ASTM D4767)
  (note: The laboratory results from the triaxial laboratory testing are still pending.)

In addition, the following analytical tests were performed by Chemax Laboratories to evaluate the potential for soil to corrode steel or to adversely react with Portland cement concrete:

- Soluble Sulfate Content
- Resistivity and pH

Individual laboratory test results can be found on the boring logs and on the plates in Appendix C, at the end of this report. It should be noted that the gradation analyses presented on Plates C1 through C27 were performed on small samples, mostly of matrix materials, and generally are not representative of the entire soil/cobble/boulder mass.

### 4.2.2 Analytical Laboratory Testing

As discussed in Section 4.1.3, the soil samples collected for analysis of man-made contaminants were analyzed first using a PetroFlag field test kit. Its utility is in qualitatively assessing the concentration of petroleum in soil, and is most beneficial when “heavy-end” petroleum products (motor oil, weathered heating oil & diesel) are present. The soil sample from each boring that contained the highest apparent concentration of petroleum, using this test method, was transmitted to Alpha Analytical Laboratory (Alpha) in Sparks Nevada for analysis of total extractable petroleum hydrocarbons by modified EPA Method 8015 and volatile organics by EPA Method 8010. Alpha is accredited in the State to perform these analyses. These methods of analysis were selected based on known widespread subsurface plumes of chemicals identifiable by these methods in the downtown Reno area. In addition, the modified 8015 Method is used by the State of Nevada to discriminate between excessively contaminated and non-excessively petroleum contaminated soil in assessing the need for remediation and offsite disposal limitations/requirements.

Groundwater samples collected from the monitoring wells and drill stems were also analyzed by Alpha Analytical, Inc. They analyzed these samples by the same methods as were the soil samples (modified EPA Method 8015 and EPA Method 601)

Results of the above described analytical tests on soil and groundwater are summarized on Tables 1 and 3, respectively, in Appendix D. Analytical laboratory reports and chain-of-custody forms are also included in Appendix D.

# 5. FINDINGS OF CURRENT INVESTIGATION

## 5.1 Site Conditions

Our field investigation for the mainline alignment was conducted along the north and south sides of approximately 2.1 miles of existing Union Pacific right-of-way between West 2nd and Sutro Streets. Borings for the western third of the project alignment (West 2nd Street to Ralston Street) were drilled along existing surface streets and in the parking lots and open spaces between the surrounding businesses. Businesses along this section of the alignment were predominately industrial in nature, consisting of a lumber yard, several auto repair services, Eagle Window & Door Company, America Rents, Reno Fuel Company, Reno Iron Works, Silver State Elevator Company, etc. Non-industrial businesses included the Burger King and Center Street Mission west of Keystone Avenue, and the Studio 3 Apartments, Sarazin Arms Apartments and St. Vincent Mission between Washington and Ralston Streets. The Truckee River meanders to within approximately 150 feet of the centerline of the existing Union Pacific right-of-way, east of the intersection of West 2nd Street and Dickerson Road.

Borings for the middle third of the alignment (Ralston Street to Evans Street) were drilled in the existing surface streets, predominately along Commercial Row and in the parking lots of the surrounding businesses. The surrounding businesses are service and entertainment oriented, consisting of downtown hotel/casinos (The Colonial, Sands Regency, Kings Inn, Eldorado, Flamingo Hilton, Fitzgerald's and Harold's Club), parking structures for the Flamingo Hilton and Harrah's Hotel/Casino the National Bowling Stadium and several small bars and clubs. The historic Amtrak Station is also located downtown just west of Lake Street. There are two pedestrian walkways over the existing railway right-of-way. The first overpass is located south of the Sands Regency, between Arlington Avenue and Ralston Street. The second overpass is the Fitzgerald's walkway, west of Virginia Street.

Borings for the eastern third of the alignment (Evans Street to Sutro Street) were drilled predominately in the open spaces between the surrounding industrial businesses. Businesses included R Supply Company, ABF Freight System, Inc., Martin Iron Works, Desert Glass Company, Reno Forklift, and others. There is a Sierra Pacific Power Company substation located along the alignment approximately 1,500 feet east of Evans Avenue. The Truckee River meanders just west of the substation to within approximately 200 feet of centerline of the Union Pacific right-of-way.

The ground surface along the proposed alignment is generally level with a slight overall slope of less than 1% towards the east. The existing alignment has a total relief of approximately 55 feet in the study area. Drainage in the industrial areas consists of normal sheet flow to drop inlets along the surface roads, or to low spots in undeveloped areas, or to the Truckee River. Drainage within the downtown area consists of normal sheet flow to drop inlets.

Site conditions for the shoofly borings are similar to conditions of mainline alignment borings. The shoofly borings were advanced within 300 feet south of the Union Pacific right-of-way. Borings for the western section of the shoofly were drilled at the ends of several surface streets, within a lumber yard and along an alley way running parallel to the existing railway line, between the railway and 2nd Street. Only a single shoofly boring was drilled in the middle third of the alignment due to access and the numerous borings already drilled for the mainline alignment along Commercial Row. The single boring was advanced in a City of Reno Parking lot, next to the Old College building, west of Arlington Avenue. Borings for the final third were drilled in open spaces along the Truckee River and in yards of the surrounding industrial businesses.

## 5.2 Subsurface Soil Conditions

### 5.2.1 General

The following paragraphs summarize the results of our field exploration. The boring logs should be reviewed for a more detailed description of the subsurface conditions at the locations explored.

### 5.2.2 Railway Corridor Borings

We encountered Quaternary Tahoe Outwash materials consisting of interbedded, discontinuous layers of sand and gravel with clay and silt matrices, clean sand and gravel, sandy silt, silt, sandy clay and clay. Many of the subsurface layers include granite, andesite and basalt cobbles and boulders up to 6 feet in diameter, in a matrix of silty sand.

Fill soils were encountered in 19 of the 33 main railroad corridor and bridge borings. The fill soils consisted of sandy clays, clayey sand, silty sand and silty gravel and varied in thickness from 2 feet to 18 feet. The largest quantities of fill were encountered in borings B25 and B28 located on the north side of the existing rail line just east of Lake Street and West of Wells Avenue, respectively. It is possible that fill was present in many of the other borings, however, no anomalies or debris were observed to demarcate the material from native soils.

A near surface layer of fine-grained material (silt and/or clay) and sand was noted in 13 of the 33 borings. These layers varied from approximately 2 feet to 17 feet thick. The thickest layers were encountered in Borings B16 through B19, located along both the north and south sides of the existing railway corridor between Arlington and Sierra Streets.

In the three large diameter borings, we made visual estimates of the amount of cobbles (3 inches to 12 inches in diameter) and boulders (greater than 12 inches in diameter) encountered during drilling operations. The subsurface conditions were variable. In large diameter boring LG1, cobbles and boulders made up approximately 30% to 40% of the total volume of materials removed between depths of 4 feet to 28 feet. Boulders 2 feet to 3 feet in diameter were removed during drilling. No cobbles or boulders, but only coarse gravel up to 1 inch in diameter was observed deeper than 28 feet. In boring LG2, the majority of the strata contained approximately 30% to 40% cobbles and boulders. A single granite boulder, 2 feet in diameter, was cored at approximately 26 feet to 28 feet in depth. In LG3, a layer from approximately 18 feet to 32 feet contained and estimated 30% to 40% cobbles and boulders. The amount of oversized material (greater than 3-inch diameter) decreased to approximately 20% from 32 feet to 50 feet with no oversized material observed below 50 feet.

#### 5.2.3 Shoofly Alignment

Subsurface conditions along the shoofly route consist predominately of medium dense to very dense layers of sandy clay and clayey sand with gravel, cobbles and boulders. The majority of the shoofly borings encountered practical refusal in cobbles and boulders at less than 5 feet in depth. These materials should provide good to very good subgrade support for rail traffic loads.

#### 5.2.4 Groundwater Considerations

We have reproduced a portion of the Reno Folio Hydrologic Map (Colby et al.) showing the historic depth to water table, using 20-foot contours on Plate 1, Appendix G. This map indicates depths to groundwater varying from less than 20 feet at the far west end, to between 20 and 40 feet throughout the majority of the corridor. Groundwater was encountered during our exploration, performed during the period of the year with generally low groundwater levels, at depths ranging from 14 to 31 feet below the ground surface. Groundwater levels were subsequently monitored in 13 of the borings completed as wells, along with 2 wells constructed by Washoe County. These levels are shown on Table 4A below. The shallowest groundwater levels were generally noted in the March and April readings, with the deepest generally in the August readings. Typical yearly seasonal variations of 2 to 4 feet were noted, with ranges of seasonal variations from 1.3 to 6.4 feet, as shown on Table 4B. In general, the greatest noted fluctuations occurred at locations closest to the Truckee River, as would be predicted.

## TABLE 4A

GROUNDWATER LEVELS
CENTRAL PART OF CORRIDOR

| Well Number (West to East) | Well Casing Elevation | DTW (8/99) | DTW (9/99) | DTW (10/24/99) | DTW (12/16/99) | DTW (3/17/00) | DTW (4/27/00) |
|---|---|---|---|---|---|---|---|
| B14 | 4513.5 | 32.7 | 30.6 | 29.3 | 29.5 | 26.3 | 27.2 |
| B19 | 4503.0 | 31.5 | 28.8 | 28.7 | 28.7 | 26.6 | 27.0 |
| B20 | 4497.8 | 27.8 | 25.9 | 26.3 | 26.4 | 25.4 | 25.4 |
| B22 | 4495.2 | 26.0 | 24.7 | 25.2 | 24.8 | 24.4 | 24.4 |
| B24 | 4491.5 | 25.3 | 23.2 | 23.4 | 26.4 | 22.9 | 22.7 |
| P1 | 4487.8 | 22.6 | 21.1 | 21.5 | 21.5 | 20.5 | 20.3 |
| P2 | 4486.6 | 23.0 | 20.5 | 20.6 | 20.6 | 20.3 | 19.0 |
| P3 | 4489.1 | 25.2 | 25.3 | 26.2 | 26.2 | 24.4 | 23.9 |
| B26 | 4489.0* | -- | -- | 20.0 | 19.8 | -- | 18.7 |
| B27 | 4486.3 | 25.6 | 22.7 | 23.7 | 23.7 | 22.0 | 21.4 |
| B28 | 4490.4 | 28.6 | 29.8 | 30.5 | 30.5 | 29.0 | 28.4 |
| B29 | 4487.6 | 34.6 | 33.0 | 33.3 | 33.0 | 32.3 | -- |
| B30 | 4487.9* | -- | -- | 27.9 | 27.9 | 27.9 | 27.9 |
| WCS | 4476.5 | -- | -- | 14.2 | 14.4 | -- | -- |
| WCD | 4476.2 | -- | -- | 49.7 | 38.1 | -- | -- |

*Casing elevations are estimates.
DTW = Depth to Water

**TABLE 4B**

GROUNDWATER FLUCTUATIONS
BETWEEN AUGUST 1999 AND MARCH 2000

| Well Number (West to East) | Maximum Fluctuation in GW Level (feet) |
|---|---|
| B14 | 6.4 |
| B19 | 4.9 |
| B20 | 2.4 |
| B22 | 1.6 |
| B24 | 2.6 |
| P1 | 2.3 |
| P2 | 4.0 |
| P3 | 1.3 |
| B27 | 4.2 |
| B28 | 2.1 |
| B29 | 2.3 |

At the proposed 1% grade, the excavation for the portion of the proposed alignment between approximately Ralston Street and the SPPCo substation site located west of Wells Avenue, may extend below the groundwater table during periods of high groundwater. Eleven wells were installed within this portion of the alignment. The proposed top of rail elevation at each boring location is shown in Table 4C along with recommended design groundwater levels.

**TABLE 4C**

GROUNDWATER FLUCTUATIONS

| Well Number (West to East) | Approximate Top of Rail Elevation (a) | Design Groundwater Elevation |
|---|---|---|
| B14 | 4492.5 | 4490.5 |
| B19 | 4480.0 | 4479.0 |
| B20 | 4473.0 | 4475.0 |
| B22 | 4470.5 | 4474.0 |
| B24 | 4469.0 | 4471.5 |
| P1 | 4470.0 | 4470.0 |
| P2 | 4471.5 | 4470.0 |
| P3 | 4473.0 | 4468.0 |
| B27 | 4474.0 | 4467.5 |
| B28 | 4475.0 | 4464.0 |
| B29 | 4478.0 | 4462.5 |

(a) Using Profile for Alternate 2

Plate 2 in Appendix G shows a profile of the planned trench along with our best information regarding high and low groundwater levels between August, 1999 and April, 2000. Plates 3 and

4 in Appendix G presents a groundwater contour map based on information developed for this study. Fluctuations in the level of the groundwater and soil moisture conditions as noted in this report will occur because of variations in precipitation, land use, irrigation and other factors.

Plates 5 and 6 in Appendix G show groundwater flow maps generated using data obtained before and after the pumping test. These maps show an overall easterly hydraulic gradient. Plate 7 in Appendix G shows a hydrogeologic cross section in the area of P-2 pumping test. This section suggests that the overall hydraulic gradient is towards the east but there is also a southern component towards the Truckee River. In addition, it suggests that at the time these data were collected the Truckee River was a "losing stream" in this area with a hydraulic head higher than observed in the adjacent water table aquifer. Municipal supply pumping is believed to have caused water level declines in the deep aquifer horizon monitored by well 7NS (also known as WCS) because it continued to exhibit drawdown for at least eight hours after test pumping in well P-2 stopped.

The hydraulic conductivity and transmissivity of the subsurface materials were first measured in the monitoring wells using the slug tests described earlier in this report. The results of these tests are presented in Table 2 (Page 22); graphs and analytical output are shown in Appendix E.

The hydraulic conductivity and transmissivity of the subsurface materials were also assessed by a constant pumping rate aquifer performance test which was conducted in pumping well P-2 using the method described earlier in this report. The results of this test are presented in the preceding Table 3 and analysis graphs are shown in Appendix F. The various aquifer test analytical methods are dependent on assumptions of infinite lateral extent of the aquifer, horizontal flow and aquifer homogeneity and isotropy. These assumptions are rarely met in practice and were not satisfied in the complex depositional environment of the pumping test site. The best approach under these conditions is to apply the several analytical methods to the individual wells and report the resulting range of hydraulic parameter values.

The U.S. Geological Survey MODFLOW model was applied to reproduce the aquifer performance test using the actual water withdrawal schedule recorded during the test. The model was stochastically calibrated to reproduce the observed drawdowns at the end of the test in three of the four observation wells. Hydraulic parameters evaluated from this model are included in Table 3, Page 24

Based on the slug tests, the hydraulic conductivities range from 2.5 ft/day to 1200 ft/day. The constant pumping rate aquifer performance test yielded hydraulic conductivities from 100 ft/day to 690 ft/day. Literature values for the hydraulic conductivity of clean gravel and sand mixtures ranges from 3 ft/day to 3000ft/day. This range correlates reasonably well with on-site soil classifications consisting of gravel with varying amounts of sand and clay. However, there does not appear to be a close correlation between the measured hydraulic conductivities and the observed amount of sand and clay within the subsurface materials. This observation suggests that the hydraulic response of the wells is largely contributed by relatively thin flow zones or horizons with relatively high transmissivities. Evaluation of the sonic drill core from well P-3 indicates that the most productive gravel beds may constitute one-third to one-fourth of the full thickness of the 20 foot screened interval in this well.

The aquifer performance test (APT) generated data that is representative of a large volume of aquifer material, and is a more reliable indicator of groundwater conditions than slug tests. Using the APT data, the bulk hydraulic conductivity averaged over a 30-foot thickness of saturated sedimentary materials was assessed to range from 100 ft/day to 690 ft/day. (The bulk hydraulic conductivity is representative of a large volume of porous media surrounding the pumped well and extending to at least the monitor wells. This term is used to distinguish the pumping test results from the hydraulic conductivity values determined from slug tests which are sensitive to a very limited volume of aquifer in the immediate proximity of the borehole.) Considering the stratified nature of the glacial outwash and paleo-river channel deposits, individual gravel beds may exhibit permeabilities 3 or 4 times greater than the bulk values. Therefore, hydraulic conductivities as high as 2800 ft/day may occur in individual gravel beds. Vertical hydraulic conductivity is typically two to three orders of magnitude less than the horizontal hydraulic conductivity in sedimentary sequences with high silt content. In an individual gravel bed, however, the vertical conductivity may equal the horizontal conductivity.

The hydraulic conductivity data can be applied to predict the groundwater flow into an excavation expressing them in terms of gallons of groundwater flow per square foot of exposed excavation face per foot below the water table level. The hydraulic conductivity parameter is simply a proportionality constant between rate of flow and hydraulic gradient. The assessed hydraulic conductivity of the subsurface materials at the site range from 2.5 ft/day up to approximately 2800 ft/day. In American practical units, this range corresponds to initial flow rates of 20 gallons per day to 21,000 gallons per day (<1 gpm to 15 gpm) per square foot of excavation face per foot of depth below the water table. Long-term excavation inflow rates would be less than these maximum rates because the formation would become dewatered and the saturated thickness and hydraulic gradient would rapidly decline. Actual inflow rates will be site specific and highly dependant on the number and lateral extent of gravel beds encountered in excavations below the water table.

## 5.3 <u>Laboratory Test Results</u>

### 5.3.1 Laboratory Testing of Soils

#### *5.3.1.1 Geotechnical Test Results*

Laboratory testing was performed as previously discussed in Section 4.2. The test data were evaluated in combination with our field exploration information to assess the engineering properties of the predominant soil types. The detailed test results are included in Appendix C.

*Classification Test Results*:
Our laboratory testing included gradation analyses and washes (% passing No. 200 sieve size) on 95 soil samples. A statistical analysis of these results indicates the amount of fines (passing the 200 sieve) generally decreases with depth. Our statistical analysis indicates an average of 33% fines from 0 feet to 10 feet, 27% fines from 10 feet to 20 feet, 16% from 20 feet to 30 feet and 14% from 30 feet to 40 feet. However, there is great variability at all depths, with frequent clean sand, open gravel, silt and clay layers. Standard deviations for the depths explored vary from 13% to 33%. Also, the results are somewhat skewed since laboratory testing was performed only on materials passing the 3-inch sieve. The results do not take into account the total percentage by weight of cobbles and boulders, which, as discussed earlier, is typically in the range of 20% to 40%.

*Moisture Contents and Dry Density:*
In-situ moisture contents and dry densities were measured on 87 and 33 select samples, respectively. In-situ moisture contents and dry densities are provided on the borings logs in Appendix A. Density testing could be performed only on 2-inch inside diameter drive samples taken during ODEX drilling operations. In zones where the logs note coarse gravel, cobbles and boulders, the recorded values should be considered only to reflect the dry density of the matrix soil materials.

*Atterberg Limits:*

Plasticity testing on samples of the clay strata and clay matrix materials indicates the clays are generally moderately plastic with Plasticity Indices in the range of 15 to 25. However, a single clay sample from Boring B-7 at 17 feet had an extremely high plasticity index of 90. This material appears to be an anomaly and was not observed in any of the other borings. See Appendix C for these results.

*Soil Strength:*
Direct Shear testing was performed on nine select driven relatively undisturbed tube samples. The results indicate a large variability in the subsurface strength parameters. A summary of our test results are provided in Table 5. More detailed laboratory results for direct shear tests are provided in Appendix C.

**TABLE 5**

RESULTS OF DIRECT SHEAR TESTS

| Boring No. | Sample Depth (ft.) | Sample Description | Friction Angle (degrees) | Cohesion (psf) |
|---|---|---|---|---|
| B-6 | 5 | Brown Sandy Silty Clay (CL) | 7 | 900 |
| B-9 | 8 | Gray Brown Sandy Gravel (GP) | 43 | 0 |
| B-14 | 38.5 | Gray Silty Gravelly Sand (SP/SM) | 49 | 160 |
| B-15 | 55 | Dark Brown Silty Sand (SW/SM) | 39 | 0 |
| B-18 | 11.5 | Light Brown Sandy Silt (ML) | 31 | 0 |
| B-18 | 21 | Brown Silty Sand (SM) | 17 | 0 |
| B-27 | 28 | Brown Clayey Sand (SC) | 16 | 400 |
| B-32 | 15 | Brown Sand (SP) | 33 | 0 |
| P1 | 40 | Brown Silty Sand (SM) | 36 | 500 |

Results of Triaxial testing, multistage, isotropic, consolidated undrained triaxial tests with pore pressure measurements were performed on three selected soil samples. A summary of the test results are show in Table 6.

## TABLE 6

RESULTS OF TRIAXIAL TESTS

| Boring No. | Sample Depth (ft.) | Sample Description | Total Stress | | Effective Stress | |
|---|---|---|---|---|---|---|
| | | | C (ksf) | ∅ (degrees) | C′ (ksf) | ∅′ (degrees) |
| B-18 | 16 | Brown Silty Sand (SM)* | 0.4 | 20 | -- | -- |
| B-18 | 40.5 | Brown Sandy Gravel (GM)** | 0.5 | 38 | 0 | 42 |
| B-32 | 16 | Brown Sand (SP) | 0.4 | 40 | 0 | 41 |

* effective stress values deleted due to apparent pore pressure equalization difficulties during testing

** test performed on finer matrix material

*Permeability:*

Eight constant head permeability tests were performed on relatively undisturbed drive samples. In areas where the soils are logged as predominately gravel, these tests should be considered to reflect the permeability of the soil matrix. A summary of our test results is provided in Table 7.

## TABLE 7

RESULTS OF LABORATORY PERMEABILITY TESTS

| Boring No. | Sample Depth (ft.) | Sample Description | k, cm/sec | Backpressure, psi |
|---|---|---|---|---|
| B-8 | 7 | Brown Sand (SP) | $1.1 \times 10^{-02}$ | 0 |
| B-15 | 15 | Brown Gray Sandy Gravel (GP) | $1.7 \times 10^{-02}$ | 0 |
| B-15 | 30 | Red Brown Clayey Gravel (GC) | $3.1 \times 10^{-06*}$ | 0 |
| B-18 | 15 | Light Brown Sandy Silt (ML) | $2.4 \times 10^{-06}$ | 0 |
| B-18 | 30 | Gray Brown Sandy Gravel w/ Silt (GM/GP) | $2.2 \times 10^{-07*}$ | 6 |
| B-19 | 38.5 | Brown Gravelly Sand w/ Silt (SP) | $2.2 \times 10^{-05}$ | 0 |
| B-27 | 23.5 | Brown Silty Sand (SM) | $1.2 \times 0^{-08*}$ | 10 |
| B-28 | 25 | Gray Brown Silty Sandy Gravel (GM/GP) | $1.1 \times 0^{-02}$ | 0 |

* k-Values not believed to represent entire soil mass; actual permeabilities believed to be 1 to 3 orders of magnitude faster.

Listed in Table 8 are a range of estimated permeability values for the subsurface conditions encountered during our investigation. These values are based on gradation results and an assumed specific gravity of 2.55. The estimates were calculated in accordance with recommendations provided in the Federal Highway Administration Report "Highway Subdrainage Design", No. FHWA-TS-80-224, 1980. The estimated range of values is shown by soil type as defined by Unified Soil Classification System.

## TABLE 8

CALCULATED SOIL PERMEABILITY BASED ON GRADATION

| Soil Type | Permeability Values (k) (cm/sec) |
|---|---|
| CL/CH (clay) | 1.1 x $10^{-5}$ to 3.9 x $10^{-7}$ |
| SP/SW (clean sand) | 3.1 x $10^{-3}$ to 7.9 x $10^{-4}$ |
| SC/SM (clayey to silty sand) | 1.0 x $10^{-4}$ to 2.1 x $10^{-8}$ |
| GP/GW (clean gravels) | 1.7 x 10′ to 5.8 x $10^{-4}$ |
| GC/GM (clayey to silty gravel) | 3.8 x $10^{-4}$ to 1.2 x $10^{-6}$ |

*Consolidation:*

We performed a single odometer consolidation test on a clay sand material from Boring B27 at 32.5 to 34 feet, Plate 52, Appendix C. Our consolidation testing was limited to drive tube samples taken during ODEX/Stradex drilling operations. The quality of the drive samples and the amount of granular material limited our selection of samples. The test was performed on a 2 inch diameter by 1 inch high specimen. The consolidation curve is presented in Appendix C.

*Concrete Reactivity and Corrosivity:*

Analytical testing was performed on five selected soil samples to assess the potential for adverse reactivity with concrete and corrosivity with steel. Soluble sulfate tests were performed to evaluate potential sulfate attack of Portland cement concrete. As shown in Appendix C, Plates 54 and 55, soluble sulfate contents were observed to be 20 to 300 parts per million (ppm).

Resistivity tests are used as an indication of possible corrosion activity. Generally, the lower the native resistivity of the soils, the more likely that galvanic currents may occur and corrosion result. Four of the five soil samples tested had resistivity of 2160 to 17500 ohm-cm (Appendix C, Plate 53) and, therefore, appear to have a fair to excellent corrosion resistance where metal will be in contact with native soils. A single sample from Boring B-16 at 6.5 feet had a resistivity value of 646 ohm-cm indicating high corrosion potential.

*Subgrade Resistance Value:*

R-value tests were performed on three samples of near surface samples of sandy clay, gravelly sand and silty sand with gravel encountered along the proposed shoofly alignment. Test results indicate R-values at 300 psi exudation pressure of 5, 68 and 57, respectively (see Plates 56-58, Appendix C).

#### *5.3.1.2 Analytical (Contaminant) Test Results*

Analytical data obtained from the analysis of soil samples collected during the installation of borings along the present rail alignment are summarized in Appendix D. Copies of original data sheets and chain-of-custody forms are also presented in Appendix D. Data presented in Table 1, Appendix D indicate that none of the samples analyzed by Alpha Analytical Inc. contained a detectable concentration of volatile organic compounds, identifiable by EPA Method 8010. Nineteen out of thirty samples contained a detectable concentration of total petroleum hydrocarbons (TPH) identifiable by modified EPA Method 8015 extractable. Of these nineteen samples, five samples (from Borings B-2, B-7, B-9, B-32, & P-1) contained a concentration of petroleum in excess of the state of Nevada action level of 100 mg/kg. Only one sample analyzed from boring P-1, contained a concentration of TPH of 820 mg/kg, which exceeds the maximum permitted concentration of TPH in soil allowed for disposal at the local Lockwood Landfill.

### 5.3.2 Laboratory Testing of Water

Table 2, Appendix D presents the analytical data from groundwater samples collected from borings and monitoring wells. These data indicate that all of the samples analyzed contained a detectable concentration of either dissolved petroleum hydrocarbons or chlorinated volatile organic compounds. In general terms the concentrations of the contaminants identified in the in groundwater samples were less than 20 µg/l for total chlorinated volatile organic compounds, and less than 10 mg/l of dissolved petroleum products. It should also be noted that none of the 11 monitoring wells installed and sampled during this project contained free phase (floating on water) petroleum.

# 6. SUMMARY AND DISCUSSION

## 6.1 Geologic and Geotechnical Conditions

Information obtained from our review of past geotechnical reports, construction projects, and published literature is consistent with data gathered in the current study. Reno is located between the Sierra Nevada geologic province to the west and the Basin and Range geologic province to the east. The Truckee Meadows is one such basin. The downtown Reno area in the vicinity of the existing railroad main line is underlain by fine to coarse grained glacial outwash, often capped by a 2 to 18 foot thick layer of finer grained fill or flood plain silt/sand deposits.

The glacial outwash unit, known as Tahoe Outwash, was formed as glacial meltwaters flowed from the Sierra Nevada mountains down the Truckee River, depositing a heterogeneous mixture of soils ranging from fine grained to very coarse grained. Tahoe Outwash predominately contains silty to clayey sandy gravel, containing frequent cobbles and small boulders up to 18 to 24 inches in diameter and occasional giant boulders on the order of 6 to 8 feet in diameter. These large boulders were deposited during periods of catastrophic flooding during glacial melt periods, as ice dams of Lake Tahoe were breached

Reno is set in a seismically active area, and the rail corridor will likely be subjected to at least moderate seismic shaking in the next 50 years or so. However, since no known action faults cross the rail corridor, this seismic shaking will likely originate from faults well outside the corridor study area. No geologic hazards including ground rupture, rockfalls, landslides, or liquefaction have occurred in historic time within the corridor, and there is no evidence that these have occurred within recent geologic time.

In general, the surface veneer of fill soils can be characterized as having moderate shear strength, low to moderate compressibility (potential for detrimental post-construction settlement), low to moderate pavement subgrade support characteristics, and low to moderate permeabilities. In general, the underlying native soils can be characterized as having moderate to high shear strength, low compressibility, moderate to high subgrade support characteristics, and low potential to adversely react with concrete or corrode steel. However, one sample tested showed a high corrosion potential. We recommend that, if used, permanent tiebacks be corrosive resistant, using epoxy-coated stressing steel and high corrosion protection.

## 6.2 Groundwater Conditions

Information obtained from the current study is fairly consistent with and supports the findings of previous investigations, published literature, and available information from public sources. Depth to groundwater is in the range of 20 feet to 40 feet in most parts of the railroad corridor. The groundwater depth shallows where the Truckee River is at its closest points to the railroad, both at the western end and between Valley Road and North Park Street, where groundwater depths of 15 feet to 20 feet occur. Seasonal variations in groundwater depth on the order of 2 feet to 4 feet are typical, and maximum historical variations on the order of 6 feet may be assumed. At the proposed 1% grade for the depressed trainway, the planned excavation is expected to encounter groundwater between approximately West Street and the Sierra Pacific Substation, west of Wells Avenue. During periods of very high groundwater levels, the completed trench may be expected to extend below the elevation of the bottom of the trench between about Ralston Street and the SPPCo Substation.

Groundwater flows in downtown Reno predominately in a west to east direction, parallel to the Truckee River. Along the railroad alignment, a southern component also exists for the hydraulic gradient, towards the river. However, local dewatering by downtown businesses causes localized deviations from this regional pattern.

Downtown Reno is underlain by groundwater which contains both: 1) manmade chemical contamination in the form of volatile organic components and petroleum hydrocarbons, and 2) elevated levels of naturally occurring chemicals that may have been more highly concentrated due to man’s past activities, including nitrogen, phosphorous and total dissolved solids. Prior to discharge of any groundwater to either the sanitary sewer or the Truckee River, significant and costly treatment is required.

Hydraulic conductivities measured in the lab and field vary greatly, as would be expected given the heterogeneous nature of the materials encountered. Hydraulic conductivities or “*k*” values range from $10^{-1}$ to $10^{-5}$ cm/sec, with typical values in the range of $10^{-1}$ to $10^{-2}$ cm/sec, equivalent to approximately 200 feet to 600 feet per day. These rapid flow rates indicate that to effectively dewater trench excavations, a very large volume of water would be generated. Given the high cost to treat the high volumes of groundwater before discharging to the sewer or river, it appears to be essential to design and construct the trench or tunnel sides and bottom to be as watertight as is practicable.

## 6.3 Environmental Issues

A thorough discussion of soil and groundwater contaminant issues is presented in Appendices H & I. In general, nearly two-thirds of the soil samples collected contained total petroleum hydrocarbons. It should be noted that those samples tested were generally selected because of a suspicious odor or vapor reading during field screening.

Based on these data it appears that there would be significant cost savings in segregating excessively contaminated soil during most large scale construction activities within the rail corridor. It is anticipated that the material containing a petroleum concentration that is less than 100 mg/kg may be used as designated fill at other non-sensitive locations. The majority of soil that has an excessive concentration of petroleum may be locally land farmed and remediated or transported to the local landfill. The remaining soil would most likely require some type of off-site transport and remediation. To be cost effective these soils should be screened to remove large rocks (greater than 3 inches) from the waste stream.

All groundwater samples contained either or both petroleum hydrocarbons chlorinated volatile organic compounds.

The data indicate that groundwater produced during construction activities would require some type of treatment to remove manmade chemicals. Treatment systems that can manage the chemicals observed are readily available, and they should be selected based on the volume of water concentrations of anticipated chemicals as well as other factors.

Based on these data it is anticipated that some form of groundwater treatment will be required if the project construction involves dewatering. However, given the potential high flow rates and need to treat groundwater, we recommend against dewatering, and instead recommend using construction methods to minimize inflow into the trench both during construction and for long term maintenance conditions. Treatment options may include the use activated carbon, ozonation, or UV peroxidation among others. Some factors to be considered in selection of a water treatment technology include the following: ability to effectively manage a variety of contaminant types within as wide a range of influent flow rates and concentrations as practicable; unit costs for treatment; ease of operation and mobility of the system.

Following treatment, water disposal may be accomplished either through discharge to the Truckee River or into the sanitary sewer. Discharge to the river would need to be performed under a National Pollution Discharge Elimination System (NPDES) permit. Permitting would be under the oversight of the NDEP. In the recent past these types of permits for construction operations in the Reno area have been difficult to obtain, since many of the water quality parameters are more stringent than present drinking water standards. Thus the treatment system must meet or exceed present state of the art, as well as be able to remove excessive concentrations of other parameters, such as suspended sediment, nitrogen, phosphorous just to name a few.

Water may also be discharged to the sanitary sewer through a permit with the City of Reno. The influent requirements for this type of discharge are less stringent than those imposed under a NPDES permit described above. However, it should be anticipated that treatment and monitoring activities will be required prior to discharge to the sewer. In addition, the discharge volume may be limited to the available capacity of the sewer system from the point of discharge.

### 6.4 Construction Methods

The results of our field investigation indicate that the success of the Reno Railroad Corridor construction method selected must in large measure be driven by groundwater flow and quality issues as well as constructability in the native cobble/boulder materials. Numerous construction methods were investigated based on interviews with contractors and equipment suppliers, the team's experience with similar projects, the materials encountered in the borings, and the difficulties encountered during the field drilling operations for the large diameter boreholes. Construction methods must (and have) considered the following:

- Excavating through large boulders and cobbles located randomly throughout the alignment.
- The lateral and vertical variability of the soil.
- The difficulty of excavating below the groundwater table, due to contaminant and dewatering issues.
- The potential for long-term maintenance costs, unless seepage is adequately addressed.

- The close proximity of structures and their foundations adjacent to the right-of-way in the downtown area.
- The noise produced during the construction process and its effect on hotel rooms and businesses near the site.
- The length of time required to complete the construction process, particularly the central downtown portion and its shoofly.

During the field investigation it became clear that a primary criteria for selection of an appropriate construction method is the efficiency of the method to penetrate the cobbles and boulders underlying the site. The difficulty and expense of excavating boulders and cobbles must be (and have been) included in the construction cost estimates for the project. Small diameter holes (several inches in diameter) can generally be drilled efficiently, while larger diameter holes or shafts may be very slow and expensive even with large drill rigs.

The results of our hydrogeology studies indicate that conventional construction dewatering is virtually unfeasible, since it could likely require large quantities of pumping which would require treatment prior to discharge. Additionally, workers in contact with groundwater would likely require protection against skin contact (Tyvek suits). In light of this we have only considered construction methods that do not require extensive dewatering for construction below the water table.

The wall construction techniques divide into several general categories as follows:

- Walls that can be constructed in place before the trench is excavated by mixing a cementing agent with the soil or by excavation and replacement of the soil with concrete, such as the jet grouting and slurry wall methods.
- Walls constructed during or after the trench is excavated such as soil nailed, mechanically stabilized earth or conventional cantilevered walls.
- Combinations of the two such as soldier pile and lagging or hybrids of two or more methods above.

A brief summary of the methods investigated, including some methods discarded as inappropriate for the study conditions, follows. Additional, more detailed information regarding these methods of construction will also be available in the Wall and Invert Alternatives Technical Memorandum, currently being completed by Nolte.

### 6.4.1 Methods Applicable Below the Water Table

#### *6.4.1.1 Deep Mixing Method (DMM)*

Deep mixing method mechanically mixes cementitious materials such as cement or lime with soil in place. The equipment usually consists of a hollow stem shaft with mixing paddles and or discontinuous flight augers attached to the lower length. The cement is injected wet or dry through the hollow auger stem and mixed with the soil creating a column of cemented soil. The equipment can have multiple shafts mixing at one time, creating a solid wall of overlapping columns several feet in diameter, which can be reinforced with steel beams.

The soil cement mixture usually has a volume ratio of 30% to 75% cement to soils, which is also a measure of the amount of spoils. The spoils in this method solidify and can be used as fill. The cemented soil columns develop sufficient strength to retain the soils behind them when the earth on one side is excavated. Steel soldier piles can be inserted while the mix is wet for additional flexural resistance.

The deep mixing method was developed for soft, wet soils with no rocks larger than about 1/4 inch. Theoretically, if the equipment is large enough larger soil particles can be mixed and cemented. Practically, it is doubtful whether the current equipment in use can penetrate the project area's cobbles and boulders effectively and cost competitively, and pre-drilling or pre-excavation to remove boulders would likely be necessary. The surface of the DMM wall would require a pre-cast or cast-in-place finish, adding additional cost to the project. For these reasons, this method is not recommended for the Reno Rail Corridor

#### *6.4.1.2 Structural Diaphragm Slurry Wall*

A diaphragm slurry wall is constructed by a process that includes excavating through bentonite slurry that is added to the trench excavation to prevent caving. Once the excavation is completed the slurry is displaced with the addition of concrete. The excavation is controlled by a template placed on the ground surface that allows one panel at a time to be constructed. The panels are typically 3 feet thick and 8 feet long. Reinforcing steel is placed in the slurry before the concrete is placed. The panels are jointed together creating a uniform reinforced concrete wall.

Slurry walls create of spoils that must be contained and disposed of. The larger excavating equipment can fracture boulders in-place and excavate the fragments although this will decrease production rates. The rough finish of the diaphragm wall may be used as the permanent finish, thereby reducing total costs for the final wall.

#### *6.4.1.3 SPTC Wall*

Soldier Pile Tremie Concrete Walls consist of steel soldier piles placed in drilled holes and filled with lean concrete. The space between the soldier piles is excavated with a special trenching machine that cuts into the lean concrete. The trench is stabilized with bentonite slurry. The trench is then filled with tremied concrete. This process has been used extensively for deep excavations in the Bay Area particularly for the BART system.

While it provides a strong finished wall, excavation for the soldier piles and slots may be difficult due to cobbles and boulders.

#### *6.4.1.4 Jet Grouting*

Jet grouting is an injection method whereby grout is pumped at high pressure through a nozzle on the side of a rod, with a cutting bit on the end. The rod is advanced through the ground to the depth of treatment. Predrilling a small diameter hole likely would be required to advance the rod in cobbles and boulders. During withdrawal the grout is injected into the surrounding soil as the rod rotates. The intent is to create a soil cement column. Adjacent columns join to create a wall. Air and water can be simultaneously injected to enhance the effectiveness of the process.

Jet grouting is a fast treatment method, however, boulders may be difficult to penetrate without pre-drilling, leaving untreated shadows below and behind treated areas. Remedial grouting would likely be required. In addition, variations in soils will result in irregularities in the radius of grout penetration that may increase excavation costs. Reinforcement of jet grouted columns is limited to vertical bars in the rod hole. The exposed surface of the wall will require a precast or cast-in-place finished surface treatment. Jet grouting creates spoils that will solidify and can be used as fill. Jet grouting is an effective underpinning method.

#### *6.4.1.5 Secant Wall*

A secant wall system is a series of drilled cast-in-place shafts joined together by overlapping each hole. The adjacent shafts are drilled after the concrete has set sufficiently to be stiff but easily excavated in the first shaft. Alternate shafts are reinforced with steel beams to provide structural support.

Secant walls will generate a large volume of spoils, which will have to be removed. Shafts may be difficult to drill and may have to be pre-drilled to penetrate boulders. The interior face of the exposed secant wall will require a finished precast or cast-in-place surface.

#### *6.4.1.6 Sheet Pile Wall*

Sheet piles are interlocking steel sheets driven or vibrated into the ground. Alternatively, they can be placed into a slurry trench. Sheet piles can be the permanent surface of the inside face of the trench walls. The backside of the sheets in a slurry trench is filled with a soil/bentonite or cement/bentonite mixture that provides an excellent water seal.

Sheet piles cannot be driven through boulders and are very difficult to install in cobbles. A slurry trench installation for sheet piles would be the most feasible method along the corridor.

#### *6.4.1.7 Ground Freezing*

The ground freezing method uses refrigeration to freeze the water in the soil to solidify the soil and water and provide temporary support. The refrigerant is pumped through pipes installed in the ground. Once the permanent wall has been constructed the frozen soil is allowed to thaw.

Ground freezing is most applicable to projects of smaller size where groundwater is near the surface. The corridor project would require tremendous amounts of energy to freeze the ground, and thus be very expensive. For these reasons, ground freezing is not recommended for the corridor wall construction.

#### *6.4.1.8 Permeation Grouting*

The permeation grouting technique injects fluid grouts into the voids of sands and gravels to create a cemented mass. The cemented sands and gravels can be excavated vertically to facilitate the construction of a permanent wall system.

### 6.4.2 Methods Applicable Above the Water Table

#### *6.4.2.1 Soldier Pile and Lagging Wall*

A soldier pile and lagging wall consists of drilled shafts spaced typically six to eight feet apart. A steel beam is inserted into the drilled holes which are then filled with lean concrete. As the excavation proceeds the lean concrete is chipped away from the beam and timber lagging is placed behind the flange of the beams.

This method is limited by large diameter hole drilling difficulties and by the fact that lagging cannot be installed below the groundwater level. Soldier piles and lagging would require a shotcreted or formed concrete surface to finish the interior side of the wall. Shotcreted walls are likely to leak, and will therefore be limited to the ends of the corridor, when construction will not extend into the groundwater table.

If the soil has more than 15% silt or clay size fines, the grout will not flow through the voids spaces. Since a large portion of the soils are in a matrix containing more than 15% fines, this method may only have limited success, but may be useful in zones of highly permeable open gravel and cobbles.

#### *6.4.2.2 Soil Nailing*

Soil nailing is a low cost and rapid shoring method, which can also be used for the construction of permanent walls. The excavation is advanced in typically four-foot lifts and a row of near horizontal reinforcing rods or “soil nails” is installed on five to eight-foot centers. A layer of reinforcing wire mesh and rebar is placed over the face of the excavation and tied onto the soil nails. The face is then shotcreted to create the wall surface. The process is completed in one day or less for each lift. A permanent wall surface can be attached to the shotcrete surface.

Soil nailing cannot be installed below the groundwater level without dewatering. The soil nails usually extend behind the wall a distance equal to 50% to 100% the wall height and therefore caution must be used to avoid underground utilities, and easements from adjacent landowners often are necessary. Since the system is passive, strains are needed to activate the reinforcing nails. As such, deformations of between 0.1% and 0.3% of the wall height should be expected.

#### *6.4.2.3 Mechanically Stabilized Earth Walls*

Mechanically stabilized earth, MSE, walls reinforce the soil behind the wall to create a gravity structure. The soil behind the wall is excavated, replaced and compacted in thin lifts. A layer of polypropylene or other reinforcing grid is embedded in each lift. As the reinforced embankment is raised, precast concrete panels are attached to the reinforcing grid to form the face of the wall.

MSE walls cannot be constructed below the groundwater level without dewatering and they require additional space behind the wall for the reinforced embankment.

#### *6.4.2.4 Cantilevered Concrete Wall*

A concrete cantilevered retaining wall is a conventional wall that is constructed and then backfilled. The wall is supported on a spread footing that extends in front of the wall a few feet below the ground surface. The vertical segment of the wall is formed and cast-in-place with connecting reinforcing extending into the footing

A conventional cantilevered retaining wall requires a temporary slope behind the wall that may extend back a distance equal to the height of the wall. Conventional walls are not easily constructed below the groundwater level.

#### *6.4.2.5 Micro-Pile Wall*

Micro-piles are small diameter (4 inches to 12 inches); high capacity cast in place bored piles that are used to transfer load to the surrounding soil. A micro-pile wall is constructed in an A-frame configuration so the rear legs are in tension and the front legs are in compression. The piles are installed at close spacing to retain the soil between the piles. Permanent wall facing is then attached to the piles.

Because of the small diameter a large number of micro-piles are required to form a wall. The wall can only be excavated below the groundwater level with temporary dewatering or grouting behind the wall.

### 6.4.3 Combination Techniques

Since portions of the project will be above water, and others will be below, hybrid shoring systems incorporating more than one of the above techniques may offer the cost advantages.

Some systems may be limited to use above water, with the use of more expensive below water techniques. Examples of this might include the use of soil nailing with jet grouting or soldier piles with jet grouting.

### 6.4.4 Tunneling

A very preliminary assessment of a possible tunneling option indicates that conventional tunneling would require a deeper alignment extending further into groundwater, with either steeper grades or an extension of the project limits. Railroad requirements to adjacent service roads result in very large diameter tunnels. The cost of tunneling could possibly be reduced with a cover and cut technique rather than conventional tunneling. This would require construction of a soldier pile supported frame to hold up the railroad during construction. The frame could be constructed in halves permitting single-track traffic without construction of a downtown shoofly. Once the frame is constructed, trench excavation would be done with normal earth moving equipment. A cover and cut system is currently being evaluated by others on the Nolte team.

## 6.5 Bracing Systems

Stability and deformation control considerations will generally require some form of bracing system for the deeper portions of the trench. Shallow portions at the ends of the project can effectively be cantilevered or constructed as a gravity system using the MSE or soil nail concepts. Soil nails can also be used in the deeper central portion above groundwater to form a gravity wall system.

For the other shoring/wall systems, either internal bracing or external tie-backs will be necessary for support. Internal bracing would consist of cross-trench struts normally connected to horizontal waler members to distribute the strut load support to the wall. Struts tend to hamper excavation operations and fewer large struts, with larger walers, are often preferable for this reason. Permanent struts at the top could reduce the finished wall thickness and reinforcing. Once the bottom slab is poured, depending on the wall thickness and bending resistance, struts could be removed.

Tieback systems are post-tensioned anchors drilled in back of the shoring/wall, a sufficient distance beyond a theoretical failure zone, to bond the anchor for its design load. Tiebacks are attached to solder piles or walls, tested, and tensioned. Tiebacks for temporary shoring can be incorporated into bracing for permanent walls by providing additional corrosion protection, however since the length of tiebacks will most often encroach into neighboring property, this option can be difficult to implement.

## 6.6 Trench Invert Construction

As discussed earlier, construction schemes involving extensive dewatering appear to be cost prohibitive. Furthermore, the soil conditions do not lend themselves to providing an effective groundwater cutoff. With these limitations, wet construction techniques are needed to construct the trench invert. This would consist of sufficient excavation below track level to pour a tremie seal slab under water that would hold back the groundwater sufficiently to place reinforcement and pour the remainder of the invert slab in the dry. Considerable attention to detail will be required to assure that the initial seal is sufficient to limit water inflow to manageable amounts. The tremie seal pour would need to be done in a number of segments considering the length of the project In situ methods such as jet grouting, permeation grouting or lens grouting could be used in conjunction with this. Additional detail on invert construction options will be provided in the Invert Alternatives Report, currently in preparation by Nolte

# 7. GEOTECHNICAL PARAMETERS FOR DESIGN

## 7.1 General

Subsurface conditions along the route generally consist of a layer of sand, fine grained soil or fill near the surface, underlain by dense granular soils containing cobbles and boulders. The depth to the dense granular soils typically varies from the ground surface to less than 10 feet below the ground surface, with some exceptions noted below. The granular soils are normally dense to very dense with high Standard Penetration Test (N) values. Groundwater is typically in the range of 20 to 30 feet below the ground surface.

## 7.2 Lateral Earth Pressures

Lower lateral earth pressures on retaining walls and bridge abutments can result by using the on-site granular soils as backfill. It will be necessary to remove particles larger than 3 inches in size from material to be used as backfill. The lateral earth pressure exerted on the vertical structure will depend on the rigidity of the wall. If the wall is free to rotate slightly outward at the top (typically 0.005 times the height of the wall abutment, or more), the low active earth pressure condition occurs. For structures where the top of the wall is fixed, (such as a rigidly connected bridge deck/abutment), the higher at rest earth pressure condition is appropriate. The at-rest condition can be eliminated by building the wall first, allowing some movement to occur, then following with the bridge construction. Resistance to movement of a structure into the soil, is provided by passive pressure. Preliminary lateral earth pressures are provided below:

**TABLE 9**

LATERAL EARTH PRESSURES

| Lateral Earth Pressure Condition | Pressure, Pounds per Square Foot of Depth | |
|---|---|---|
| | Above Groundwater Table* | Below Groundwater Table |
| Active Earth Pressure | 35 | 80 |
| At-Rest Earth Pressure | 65 | 90 |
| Passive Pressure | 450 | 250 |

*Values assume drainage of backfill is provided

A coefficient of friction of 0.45 between the concrete foundation and the granular subsoils may be used for design.

Seismic lateral earth pressures were estimated using the Mononobe and Okabe pseudo static approach as outlined in Section 9.4 of the Federal Highway Administration Reference Manual, Publication FHWA HI-99-012. Our calculations assumed the following:

1) A horizontal backfill slope ($\beta$=0)
2) A vertical wall backface ($\theta$=0)
3) No vertical acceleration coefficient ($k_v$=0).
4) A design horizontal acceleration of 0.4 ($k_h$=0.4)
5) The angle of friction between the soil and wall is equal to half of the internal angle of friction of the soil ($\delta/\phi$=0.5)

Preliminary lateral dynamic earth pressure coefficients are provided below:

| | |
|---|---|
| Active Dynamic Earth Pressure Coefficient | $K_{ae}$ = 0.60 |
| Passive Dynamic Earth Pressure Coefficient | $K_{pe}$ = 4.95 |

The total dynamic active and passive earth pressures can calculated by the following equations:

$$\Delta P_{ae} = 1/2\gamma H^2 K_{ae}$$

and

$$P_{pe} = 1/2\gamma H^2 K_{pe}$$

where:

H = the height of the soil face (wall height in feet)
$\gamma$ = average unit weight of soils (approximately 130 pcf)

It should be noted that the total dynamic pressure calculated consists of two parts. For the active case the total dynamic pressure consists of the static earth pressure $P_a$ and the incremental dynamic earth pressure $\Delta P_{ae}$ ($\Delta P_{ae} = \Delta P_{ae} - P_a$). For the passive case the total dynamic pressure consist of $P_p$ and $\Delta P_{pe}$. The static pressure is assumed to act at H/3. The incremental dynamic pressure is assumed to act at 0.6H.

For design, we recommend a minimum factor of safety of 1.1 be used against bearing capacity failure and sliding resistance and 1.5 for overturning stability.

## 7.3 Bridge Foundation Design Parameters and Anticipated Settlement

In developing preliminary geotechnical design parameters for bridge foundations, it is assumed the structures will be of concrete beam or steel girder construction, simply supported at each end. Two options for bridge foundation support may be feasible. Structural bridge loads may be supported by independent foundation members, or on the retaining wall on each side of the railway. If loads are supported on the retaining wall, it will be necessary to design the wall section and foundation to form a footing and accommodate the combined (lateral and vertical) bridge loads.

The subsurface soil profile at most of the proposed bridge locations consists of native dense sand or gravel below depths in the range of 1 to 10 feet. This granular layer generally extends well below the proposed retaining wall section, and is overlain by looser sand, fine grained (silt or clay) soil, or fill. The exception to this condition is at the West Street bridge location (Test Boring B-18). At the West Street bridge location, fine grained soils are present within the upper 20 to 25 feet.

It appears the most feasible foundation types for the bridges would be spread footings or a mat type foundation incorporated into the trench wall system. Alternatively, separate bridge foundations, not connected to the wall, may be used. In this case, the footings should bear in the dense granular sand and gravel, at minimum depths of 5 feet and up to 10 feet. Foundations should be deepened so as not to load the trench wall, or the trench wall should be designed and reinforced to take these additional loads. Alternatively they may be set back laterally from the trench wall to be out of the zone of influence. Foundations bearing in the granular soil can be designed for a relatively high bearing pressure, and will experience low settlements. At the West Street bridge location, foundations would bear in the fine grained soil. A lower bearing pressure will be necessary in this area, unless a deep foundation system is utilized to transfer loads via piers to the dense granular layer.

*Vehicle/Pedestrian Bridges*

If bridge foundations are incorporated into the retaining wall, bridge loads would be transferred to the bottom of the wall. At this greater depth, it would be possible to increase the allowable bearing pressure over the values used at a shallower depth. Allowable bearing pressures, using a factor of safety of 3 to 3.2, are indicated below, for preliminary design of vehicular bridges over the railroad.

**TABLE 10**

GENERAL BRIDGE FOUNDATION DESIGN PARAMETERS

| Location | Approximate Foundation Depth, feet | Bearing Material | Allowable Bearing Pressure, psf | Remarks |
|---|---|---|---|---|
| All Bridge Locations Except West Street | Minimum of 5 to 10 | In-place Dense Sand or Gravel | 5,000 | Spread Footings Independent of the Wall |
| West Street | Minimum of 5 feet | In-place Silt or Clay | 3,000 | Spread Footing Independent of the Wall |
| All Bridge Locations Except West Street | At Bottom of Retaining Wall, about 10 to 30 feet Below Grade | In-place Dense Sand or Gravel | 6,000 – 12,000 | Spread Footing Incorporated into the Wall |
| West Street | Approximately 30 feet below grade | In-place Dense Sand or Gravel | 8,000 – 12,000 | Spread Footing Incorporated into the Wall |

The allowable bearing pressures indicated above may be increased by one-third for short term loading conditions, such as seismic or wind loading. For planning purposes, a moist unit weight of 130 pounds per cubic foot may be used for the in-place or recompacted sand or gravel. For the fill materials or native silt or clay, a moist unit weight of 115 pounds per cubic foot may be used.

The site is located in UBC Seismic Zone 3. Seismic loadings should be evaluated using the 1997 UBC Method as outlined in the Federal Highways Administration, Publication FHWA HI-99-012, December, 1998. We recommend using a $S_C$ soil profile type, which is applicable to a very dense soil site class with a shear wave velocity of 360 to 760 m/sec (1200 to 2500 ft/sec).

Total post construction settlement of individual footings, designed for the allowable bearing pressures indicated above, are expected to be in the range of one inch or less. Differential settlement between individual footings is expected to be about one-half of the total settlement.

*Sutro Avenue Bridge*

Preliminary plan, elevation and typical structural section sketches for the Sutro Avenue Substructure (underpass) were provided HDR. These sketches allowed for detailed evaluation of allowable bearing values and estimated settlement for the proposed structure at Sutro Street.

Surface conditions consist of 2 to 6 feet of granular fill (silty sand, sandy gravel and clayey gravel). The fill material was underlain by various layers of predominately coarse-grained glacial outwash materials consisting of dense to very dense clayey sand and gravel, and silty sand and gravel with cobbles and boulders up to 4 feet in diameter.

Due to the dense to very dense granular subsurface conditions, the most feasible foundation at this location would be spread footing or a mat foundation. At the abutment locations, retaining walls on spread footings would be likely. The central bridge pier would consist of a wall or columns supported on shallow spread foundations. Foundations should bottom on dense to very dense granular outwash materials.

Preliminary recommendations for allowable bearing pressures and anticipated settlements for Sutro Avenue substructure shallow foundations are provided below. Allowable bearing pressures provided as a function of footing width and embedment depth. Bearing capacities and anticipated settlements were calculated in accordance with AASHTO *Standard Specifications for Highway Bridges, 16$^{th}$ edition, 1996*, Section 4.4.7.

**TABLE 10A**

SUTRO BRIDGE FOUNDATION DESIGN AND SETTLEMENT

| *Depth (ft.) | Footing Width | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 4 ft. | | 6 ft. | | 8 ft. | | 10 ft. | |
| | Bearing Capacity (psf) | Settlement (in) | Bearing Capacity (psf) | Settlement (in) | Bearing Capacity (psf) | Settlement (in) | Bearing Capacity (psf) | Settlement (in) |
| 5 | 5000 | 0.2 | 5500 | 0.3 | 6500 | 0.4 | 7500 | 0.5 |
| 10 | 8000 | 0.4 | 9000 | 0.6 | 10000 | 0.7 | 11000 | 0.9 |
| 15 | 10000 | 0.6 | 11000 | 0.9 | 11500 | 1.0 | 12000 | 1.2 |
| 20 | 12000 | 0.8 | 12500 | 1.1 | 13000 | 1.3 | 13500 | 1.6 |
| 25 | 13000 | 1.0 | 13500 | 1.3 | 14000 | 1.6 | 14500 | 1.9 |
| 30 | 14000 | 1.2 | 14500 | 1.5 | 15000 | 1.9 | 15500 | 2.2 |

*Recommended foundation depths: minimum of 5 to 10 feet independent of the wall minimum of 10 to 30 feet below grade, incorporated into the wall

The allowable bearing pressures indicated above may be increased by one-third for short term loading conditions, such as seismic or wind loading.

7.4 Recommended Permanent Slope Angles

At either end of the project, there may be sufficient right-of-way and shallow enough slopes in many areas to make construction of conventional sloped sidewalls for the train trench feasible. We recommend that permanent fill and cut slopes at the west and east ends of the trainway be excavated to a maximum slope of 2H:1V. Satisfactory slope performance is primarily affected by drainage and runoff. Care must be taken that drainage is not directed to flow over slope faces. Interceptor (brow) ditches should be constructed at the tops of slopes in order to collect and divert runoff which otherwise would flow over the slope face. Slope faces should be protected against erosion resulting from direct rain impact and melting snow. Consideration should be given to permanent measures such as riprap or geo synthetics and vegetation.

7.5 Temporary Unconfined Excavations

Deep cuts of up to approximately 35 feet are proposed to construct the train trench. The use of steepened, temporary cut slopes will be needed to construct below grade structures **which are above the water table**. The following criteria have been developed and may be used for construction of temporary cut slopes adjacent to the proposed structures:

**TABLE 11**

TEMPORARY UNCONFINED EXCAVATIONS

| Temporary Slope Inclination (Horizontal to Vertical) | Maximum Height (Feet) |
|---|---|
| 0.5:1 | 5 |
| 0.75:1 | 20* |

*or depth to groundwater, whichever is less

These layback requirements may require modifications where very loose, cohesionless sands are encountered. Also, the above suggested laybacks are guidelines which may require modification in the field after the start of construction. The contractor is ultimately responsible for the safety of workers and should strictly observe federal and local OSHA requirements for excavation shoring and safety. Due to the granular nature of the surface soils, some ravelling of temporary cut slopes should be anticipated.

## 7.6 Temporary Trench Excavation and Backfill

It appears that excavation for utility trenches and drainage structures can be made using either a conventional backhoe or excavator in the existing native and fill soils. However, difficult excavation conditions should be anticipated due to the dense nature of the site soils, and the abundance of oversize cobbles and occasional 6 to 7 foot boulders. If trenches are extended deeper than five feet or are allowed to dry out, the excavations may become unstable and should be evaluated to verify their stability prior to occupation by construction personnel. Shoring or sloping of any deep trench walls may be necessary to protect personnel and provide temporary stability. All excavations should comply with current OSHA safety requirements for Type B soils in native materials, and Type C soils in fill materials. (Federal Register 29 CFR, Part 1926).

# 8. LIMITATIONS

Recommendations contained in this report are based on our field explorations, laboratory tests, and our understanding of the proposed construction. The study was performed using a mutually agreed upon scope of work. It is our opinion that this study was a cost-effective method to evaluate the subject site and evaluate some of the potential geotechnical concerns. More detailed, focused, and/or thorough investigations can be conducted. Further studies will tend to increase the level of assurance, however, such efforts will result in increased costs. If the Client wishes to reduce the uncertainties beyond the level associated with this study, Kleinfelder should be contacted for additional consultation.

The soils data used in the preparation of this report were obtained from borings made for this investigation. It is possible that variations in soils exist between the points explored. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at this site which are different from those described in this report, our firm should be immediately notified so that we may make any necessary revisions to our recommendations. In addition, if the scope of the proposed project, locations of structures, or building loads change from the description given in this report, our firm should be notified.

This report has been prepared for preliminary planning and preliminary design purposes for specific application to the Reno Railroad Corridor Project in accordance with the generally accepted standards of practice at the time the report was written. No warranty, express or implied, is made.

Other standards or documents referenced in any given standard cited in this report, or otherwise relied upon by the authors of this report, are only mentioned in the given standard; they are not incorporated into it or "included by reference," as that latter term is used relative to contracts or other matters of law.

This report may be used only by the Client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on- and off-site), or other factors including advances in man's understanding of applied science may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 36 months from its issue. Kleinfelder should be notified if the project is delayed by more than 24 months from the date of this report so that a review of site conditions can be made, and recommendations revised if appropriate.

# 9. REFERENCES

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